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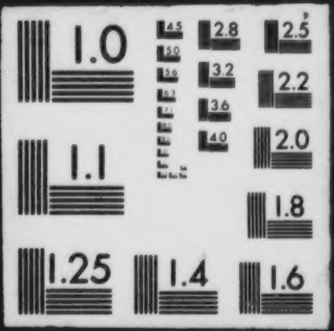
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COMBINED SEWER OVERFLOW ABATEMENT PLAN, DES MOINES, IOWA

Peter L. Davis, et al

Henningson, Durham and Richardson, Incorporated

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Several combined sewer overflow abatement projects are recommended for implementation.
(Field - EPA)

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COMBINED SEWER OVERFLOW ABATEMENT PLAN,

Des Moines, Iowa

by

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Contract No. 14-12-402
Project No. 11024FEJ

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This report was submitted for completion of Contract Number 14-12-402 between the Environmental Protection Agency and Henningson, Durham & Richardson, Inc., Omaha, Nebraska 68114.

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SECTION I

CONCLUSIONS

1. The reduction of combined sewer overflow pollution can be effected by various combinations of control and treatment facilities.

2. For particular situations, solutions incorporating both sewer separation and combined overflow treatment have considerable merit. In this study, two such situations are demonstrated by the Closes Creek system where separation is most economical and the East 18th Street System where combined overflow treatment was selected.

3. Of the various schemes investigated for treatment of combined sewer overflows, the most applicable type of treatment was impoundment. Impoundments designed for storage of storm waters are not materially affected by the rate of flow, but by the total volume received. Because of limited available land, impoundments will usually be heavily loaded and aeration may be required for treatment and for odor control. A high degree of treatment will be provided, especially to high-frequency storms. Adequate provision must be made for the removal of solids.

4. The necessity of having to handle extreme flow ranges materially increases the cost for mechanical treatment schemes. Primary treatment does not provide the degree of treatment offered by impoundments, and secondary biological processes are not generally adaptable to the intermittent nature and extreme ranges of flow encountered in storm water treatment.

5. The use of retention or retardation basins with stored wastewaters returned to the collection system permits combined sewer flows to be treated by conventional wastewater treatment facilities during off-peak periods.

6. For the reduction of pollutant discharges to the Des Moines River, expansion of the existing wastewater treatment facilities will provide the most effective use of the construction dollar. Plan B-2, which uses sewer separation in combination with interception and treatment of overflows, provides the most cost effective scheme of reducing pollutant discharges from combined sewer overflows.

7. Components of Plan B-2 can be implemented immediately to reduce the overflow of combined wastewaters and to increase the effectiveness of existing wastewater treatment facilities.

8. The existence of combined sewers creates operational problems in the treatment of domestic and industrial wastewaters and permits discharge of untreated wastewaters during wet weather conditions.

9. For metropolitan areas where the practice of constructing combined sewers was limited to early periods of growth and make up a relatively small portion of the total urban area, the organic pollutional impact from combined sewer overflows will likely be overshadowed by that from storm water discharges. The case of Des Moines illustrates this. Drainage from approximately 4,000 acres receive some quantity of combined sewer overflow and discharges an estimated 174,500 pounds of BOD annually. From the remaining 45,000 acres served by separate storm sewers, the annually BOD discharge is estimated to be 2,668,000 pounds.

10. Laboratory analyses of the discharges sampled in Des Moines showed combined sewer overflows to have slightly higher pollutant concentrations than storm water discharges, but in the same order of magnitude. Following is a summary of average pollutant concentrations as determined by the monitoring program.

SUMMARY OF AVERAGE CONCENTRATIONS, IN mg/l

Station	B.O.D.	T.S.S.	$NH_3 \cdot N$	$NO_3 \cdot N$	T.PO_4
Storm Water Discharges					
S-1	48	315	1.99	1.11	1.25
S-3	63	578	1.60	1.47	0.93
O-2	44	495	1.21	0.88	2.20
O-11	56	404	2.30	0.96	2.23
Average	53	448	1.78	1.10	1.65
Combined Sewer Overflows					
O-3	69	144	4.53	0.26	11.72
O-6	95	592	9.42	0.63	9.88
O-7	50	195	1.84	1.15	7.10
O-8	68	410	3.22	1.07	6.96
O-8A	77	303	4.94	0.57	-
Average	72	329	4.79	0.74	8.92

11. The total quantity of pollutants discharged, at least for the low and moderate intensity storms which occurred during the study, is closely related to the relative volume of flow. This is illustrated by the data in Figures 16 to 24 in Section VI of this report. For example, at Station 0-11 the relationship between the volume of runoff and quantity of pollutants discharged during three different storms is as follows:

Date	5-7-69	7-23-69	9-4-69
Rainfall - inches	1.14	0.69	0.22
Runoff - Ac-Ft	23.2	9.30	1.85
- inches	0.238	0.097	0.018
BOD - lbs/Acre	1.66	0.74	0.27
TSS - lbs/Acre	18.4	13.4	1.42

Also, as storm intensity and runoff increases, the relative percentage of sanitary sewage in the combined sewer overflow compared to storm flow decreases. Conversely, high-frequency, low intensity storms will produce overflow discharges with a high percentage of sanitary wastewater.

12. The Des Moines and Raccoon Rivers are heavily used for recreation, particularly above their confluence at Scott Street. The completion of the Saylorvill Reservior immediately above the city will enhance the Des Moines River as a recreation area. Protection of these waters for recreational use should be a consideration in the city's pollution abatement program.

13. The annual quantity of pollutant material discharged in the effluents of wastewater treatment facilities is likely to equal or exceed the quantity of pollutants in combined sewer overflows and urban storm water discharges. Wastewater treatment plant effluents may greatly exceed other sources if a high degree of treatment is not provided. The Des Moines plant, which averaged 83.5 percent BOD removal in 1970, was estimated to discharge 55 percent of the BOD, 91 percent of the nitrates, and 90 percent of the ortho-phosphates discharged from the Metro area during the year study period, exclusive of overflows or bypassing of wet weather flows.

14. Treatment plant upset by wet weather flows was not a major factor at Des Moines. This was due to (1) the ability of the trickling filter process to adjust to shock hydraulic and biological loads and (2) the hydraulic capacity limitations which restricted maximum flows to approximately 45 MGD, an increase of only 10 MGD over the average daily load of 35 MGD. Excess flows were stored in the interceptor as much as possible,

with bypassing of the plant used only to prevent property damage from basement flooding. Slightly reduced treatment efficiencies resulted from wet weather flows due to the increased hydraulic load. A high influx of suspended solids was typical.

15. A typical midwestern river, such as the Des Moines River, receives and carries enormous quantities of organic materials from rural areas. In the case studied, incoming river loads, except for phosphates, are several times the quantity of organic pollutants generated in the Metro area.

16. The relative magnitude of the river loads compared to those known to be emanating from the Des Moines urban area indicates a need for identification of their source and research to develop adequate methods for control or treatment thereof.

17. Analysis of the stream quality data indicates that the quantity of organic pollutants and nutrients which would be removed by the complete abatement of overflows is not significant compared to the present load carried by the rivers. For this reason, the need for an extensive program of combined sewer overflow abatement is not indicated.

18. High-frequency, low-intensity rainfalls produce a much lower coefficient of runoff than the values generally acceptable for design of storm drainage facilities. The results of this study indicate that for high-frequency storms the coefficient of runoff correlates more closely with the accumulated depth of precipitation than with the intensity of the precipitation. Typical values for the 0-11 Station, the 20th Street Storm Sewer, are as follows:

Accumulated Depth of Precipitation, Inches	Volumetric Coefficient of Runoff, Cy
0.2	0.10
0.4	0.15
0.6	0.19
1.0	0.25
2.0	0.33

19. The distribution of precipitation intensities with regard to both time and the total depth of precipitation can be developed from weather bureau records and provides the designer a tool for evaluating the effectiveness of storm water treatment. The procedures used in this study are described in Section VII.

20. Circumstances existing in the City of Des Moines during the course of this investigation offered an excellent opportunity for a study of this nature. The City straddles a nutrient-rich river and was found to contribute to the nutrient load as anticipated. Storm water discharges, wastewater treatment plant effluent, combined sewer overflow during periods of runoff, and overflow caused by excessive infiltration contributed to the nutrient load.

21. Analysis of the existing sewerage system to determine the magnitude of the overflow problem and for the design of corrective measures was hampered by lack of sufficiently detailed utility records. This is by no means a problem limited to the study area, but exists in a large number of cities. Utility records are often incomplete in older cities and in areas where new development has occurred without a strong central governmental authority. The preparation and maintenance of accurate records of utilities systems should be a prerequisite to any comprehensive planning program.

22. Planning for the abatement of combined sewer overflows should begin with an engineering appraisal of the relative magnitude of the problem and its impact on the receiving stream. Remedial programs, if required, should be tailored to fit each individual situation.

SECTION II

RECOMMENDATIONS

1. The first priority for the metropolitan area water pollution control program should be provision of continuous adequate treatment of domestic and industrial wastes as provided by the wastewater treatment facilities expansion program proposed for Des Moines. The expanded facility will have capacity to handle infiltration flows and a substantially greater amount of combined flows. In addition, it will provide a higher degree of treatment to all flows.

2. An immediate extensive program of sewer separation or combined sewer overflow treatment is not recommended for the City of Des Moines, based on analysis of river conditions.

3. Two components of the combined sewer overflow pollution abatement Plan B-2 can be implemented immediately to reduce overflows and provide for more effective treatment of sanitary wastewaters.

   a. The Dean Lake Impoundment designed to treat combined flows from the East 18th Street system will reduce the storm water flow to the watewater treatment plant, prevent the dilution of high strength industrial wastes from that watershed, and relieve flooding problems in the lower reach of that system.

   b. Separation of the scattered combined sewers in the Closes Creek Watershed will eliminate several troublesome overflow conditions, reduce the stormwater load in the West Side Interceptor Sewer, and prevent bacterial and organic pollutants from this source from reaching the recreation area above the Center Street Dam.

4. The City should consider other steps outlined in Plan B-2 to reduce significant and potentially hazardous overflows.

   a. Construction of the reduced Prospect Road Impoundment would eliminate all overflows from the West Side Interceptor above Grand Avenue, thus protecting the recreational area above Center Street.

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c. Frequent overflows of the Ingersoll Run Sewer could be reduced by constructing facilities to divert high-frequency overflows to the Southwest Outfall. It is estimated that approximately one-third of the annual overflow could be captured by diverting up to 5 MGD of the overflow to the Southwest Outfall in addition to that which remains in the Ingersoll Run system. This would intercept many of the high-frequency overflows.

4. Before embarking on programs for separation or treatment of combined sewer overflows, Des Moines and other cities contending with the problem of combined sewers should develop and maintain a detailed master plan for collection and treatment of all wastewaters. The plan should begin with reproducible documentation of existing sewer systems. New sewer construction can then be tailored toward the ultimate goal of the plan. It is recommended that Des Moines, in the planning of relief sewer construction, maintain the domestic and industrial wastewaters from the north and west areas of the City separate from combined flows to a point downstream of the central business district, where separate sanitary flows can be given priority to the treatment facilities.

5. To reduce the volume of extraneous and infiltration water into the sewer systems, special consideration should be given to prohibiting footing and roof drain connections to sanitary sewers. Present F.H.A. home construction criteria should be modified to insure other suitable drainage for footing drains.

6. Where treatment of combined overflow must be considered, the impoundment method is recommended wherever practical. This method is the most reliable and flexible. Stormwater flows may be retained in the impoundment for treatment or returned to the sewer system for treatment at conventional mechanical plants. Consideration must be given to loadings, and supplemental oxygen provided if necessary. Installations in residential or park areas must include landscaping. The design must include provisions for solids handling.

7. Because of the obvious imbalance noted in this study between incoming river loads and discharges from the metropolitan area, it is recommended that a program be initiated to identify all

sources of the rivers loadings and to develop adequate methods for control and/or treatment. Such a program should utilize a basin approach to the problem. To minimize duplication of effort and provide maximum dissemination of findings, river basin studies should be coordinated through a single agency.

SECTION III

INTRODUCTION

PROJECT SCOPE

This project was a multi-phasic study designed to provide engineering information regarding the volume, character and impact of combined sewer overflows and urban storm water discharges from a typical midwestern metropolitan area. The project includes an engineering evaluation of solutions to the problem, including combined sewer separation and investigation of facilities for the treatment of both combined sewer overflows and storm water discharges. In particular, the project encompasses the following:

1. Measurement, analysis and evaluation of overflow from combined sewers and discharges from selected storm sewers.

2. Analysis and evaluation of the receiving stream.

3. Development of bases of design for transport and treatment of combined sewer overflow waters.

4. Development of preliminary engineering designs and cost estimates for facilities for the interception, transmission and treatment of combined sewer overflows.

5. Evaluation of existing wastewater treatment facilities to establish maximum hydraulic and organic treatment capabilities and to determine their capabilities for treating stored or retarded combined sewer flows.

6. A detailed combined sewer separation study including maps and cost estimates for a three square mile area selected as a typical metropolitan combined sewer area.

7. Maintenance and operation of a metropolitan rain gauge network and a detailed rainfall-runoff relationship analysis for selected drainage areas.

8. Cooperation with local, regional state and federal agencies to obtain available data, to assure compliance with existing standards and regulations, and to keep them appraised of the study findings so that recommended improvements may be coordinated with their studies and planning.

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BACKGROUND

The pollution problem attributable to intense, though sporadic, combined sewer overflows and urban storm water discharges is common to many older cities, including Des Moines. Many of these cities originated as a river settlement where waterway transportation, not water pollution, was the prime consideration. Certainly the engineering problems that would one day have to be solved in order to provide adequate sewerage and storm drainage facilities were not considered in the original site selection. Often, the first sewers in the area were private sewers discharging directly to the adjacent waterway. Some of these may still be in service. Most, however, have been replaced with larger conduits and systems by community-minded people who sought to improve the environment of their community and who had the foresight to provide reserve capacity to serve the community as it grew. That these people did not foresee just how much their community would grow does not detract from their foresight and endeavor. Unfortunately, these original systems are usually now the built-up core area of the community, and almost always the most expensive area for new construction. More recent additions to the original combined system often compound the problem. Sewers which were originally constructed as separate sewers have since been combined for one or more of several reasons:

> Sanitary sewers, in areas remote from storm sewers, were tapped to provide "temporary" relief from localized flooding.
>
> Storm sewers, in areas remote from sanitary sewers, were tapped for sanitary connections.
>
> Localized overload conditions in either a sanitary or a storm sewer prompted interconnection for relief.

Unfortunately, the temporary status probably intended for the above connections has long been forgotten. That which is out of sight is too often also out of mind. Numerous cases have been noted where minor altering of the route or sizing of new sewers could have separated individual areas. Now, however, major sewer construction is often required to do the same job.

Although this study is concerned with combined sewage overflow into the receiving waters, overflow may also occur within the combined collection system. Usually such overflow discharges into natural drainageways or storm sewers and eventually reaches the receiving water. In some cases the overflow may occur at street inlets. It is possible for these waters to emerge from one inlet, run overland for some distance, and then re-enter the system through another inlet. Where this is the case, some sanitary significance may be attached to the manner in which the overflow is transported.

One culprit contributing to sanitary and combined sewer overload and overflow is the practice of connecting roof and foundation drains to the sewer. This practice is often prevalent in cities served by combined systems with the same result - imposition of a sharp hydraulic overload from roof drains followed by an extended hydraulic overload from foundation drains. During the mid-study period of this project, the study area received considerable precipitation which resulted in high short-term and considerable long-term overflow, much of which is believed to have occurred because of roof and foundation drainage.

Persons involved in water pollution control and regulation recognize combined sewer overflows and storm water discharges as pollutants of possible considerable potential. However, the magnitude of the potential and its unique behavioral characteristics have not until recently been investigated to any appreciable depth. Hence this particular study which, together with similar concurrent studies in other sections of the country, is expected to facilitate a logical, factual assessment of the problem and feasible solutions.

PROJECT DESCRIPTION

To accomplish, as best possible, the objectives of the project, it was necessary to establish and maintain a comprehensive system for monitoring and analyzing rainfall, combined sewer overflows, and storm water discharges. It was also necessary to obtain or develop extensive background data on the existing metropolitan collection system and treatment facilities. In addition, river data was obtained from limited sampling by the contractor and from other agencies. This data was needed in order to develop an accurate picture of existing river conditions.

The monitoring program involved installation and maintenance of six recording rain gauges and as many as nineteen bubbler-type water level recorders. Equipment and installation details are discussed in subsequent sections. Monitoring locations are shown in Figures 13 and 14. The rain gauges located within the study area, coupled with two U. S. Weather Bureau gauges in the area, provided a very good picture of rainfall distribution. Bubbler-type recorders were designed and assembled for security and mobility for this particular project. Although semi-permanent monitoring installations were made for numerous locations, the recorders were located at points of overflow or termini of drainage basins. However, on several occasions, bubblers were also used to record flow through wastewater treatment facilities or to determine surcharge elevations within the collection system. The surcharged sewer problem, although not a part of this study, had a profound effect on the conduct of the project and is discussed in Section XI.

Overflow and runoff sampling was generally done with commercial automatic, timer-actuated, vacuum samplers. Samples were iced during collection.

Following sample collection and a review of the runoff pattern, a decision was made whether to analyze individual samples, to prepare composites according to flow rates, or to do both when sufficient sample was available. This flexibility is one of the positive features of the particular sampler purchased for this project. Iced samples were then delivered to the contract laboratory at Iowa State University for analysis.

Concurrent with the above activities, data and maps were obtained or prepared for the combined sewer separation study. A detailed analysis for combined sewer separation, including estimated costs, is provided in the form of computer printouts and drawings. Available data on existing storm sewer systems was often incomplete and required additional field work to verify existing records. This problem is discussed also in Section XI.

Evaluation of the receiving waterways was accomplished by supplementing the data obtained from the several project samplings with considerably more extensive data available from the Iowa State Department of Health and a Corps of Engineers study being conducted by the Engineering Research Institute of Iowa State University (1). Five river samples were collected as part of this project. These samples were collected above, in and below the Des Moines Metropolitan area in order to determine season variations in the quality of the rivers. Twenty-eight hour diurnal D.O.'s were obtained during four of the samplings. Also, night-time samples for sanitary analysis were collected in order to show any variation as a result of sanitary loads. The available river data presents an interesting, though not unexpected, picture of a receiving stream which originates and courses through an area of intensive agricultural activity.

One of the key areas for investigation in this study was the feasibility and economics of treatment of combined sewer overflow as opposed to storm sewer separation. Various treatment schemes were investigated, including lagooning, mechanical treatment, chlorination, and combinations thereof with and without peak flow retardation. In addition, existing wastewater treatment facilities were evaluated in order to determine available capacity for storm water treatment during off peak periods.

The physical problems involved in collecting and transporting storm water flows are nearly as difficult, if not more difficult, than treating them. For example, because of prohibitive cost and disruption of existing utilities, streets and services, it is unlikely that significant additional hydraulic capacity will be provided through the downtown core area of Des Moines. Additional capacity could be provided in a conduit along or in the Des Moines River. This would be expensive, but feasible.

Treating stormwater, be it combined sewer overflow or separate storm water discharges, involves numerous considerations, including: (a) determination of a design storm and development of the volumetric requirements (See Section X); (b) projection of land use, population growth and future overflow and runoff; (c) determination of the capability of existing wastewater treatment facilities to handle storm waters; and (d) determination of the degree of treatment required or justified and the effectiveness of dollars spent for such treatment.

SECTION IV

PROJECT AREA

GENERAL LOCATION

Located in the heart of the North American land mass, Des Moines is the largest city in Iowa, the capitol of the State and the county seat of Polk County. In an urban area of approximately 288,000, the City of Des Moines' 1970 population was 200,600. The Study Area lies astride the Des Moines River in the central part of Iowa, as illustrated in the "Vicinity Map," Figure 1. The Raccoon River enters the City from the west and its confluence with the Des Moines River is within the City limits.

The Study Area is situated in the upper Mississippi River drainage basin. In the Des Moines and Raccoon River basins, the land is gently rolling with broad uplands and the major streams flow in narrow valleys. Des Moines and its environs cover a multiplicity of topographic features. Both rivers have relatively wide flood plains in some areas of the City and narrow or none at all in other areas. In general, drainage is from northwest to southeast with a high ridge on the left bank of the Raccoon River separating the two drainage basins so that development to the north drains to the Des Moines River. These two rivers divide the area into three sectors; the northwest, the northeast and the south. Major streams tributary to these include Walnut and Beaver Creeks in the northwest area, Four Mile and Saylor Creeks in the northeast and Middle Creek and the North River in the south area. The uplands generally lie above 900 feet elevation, with the flood plain elevation in the range of 800 feet above sea level.

Geologically, the entire area is underlain by the Pennsylvania system which forms the uppermost strata of bedrock. This system, primarily shales, also includes sandstones, coal and limestone. Overlying the bedrock are glacial deposits of various clays, sands and gravel. Wind blown loess, a uniformly fine grain soil, covers the glacial deposits. It ranges in depth of 5 to 30 feet and is generally visible where the top-soil has been removed.

Des Moines enjoys a climate which is continental in character, with cold dry air in the winter and warm moist air during the summer. A marked seasonal contrast in both temperature and precipitation is characteristic of the climate. The average annual temperature is 50 degrees, but the average temperature for the individual months varies from 21 degrees in January to 76 degrees in July. Extremesvary from an extreme low of -30 degrees in mid-winter to an extreme high of 110 degrees in mid-summer. The winter season, lasting about 19 weeks from mid-November

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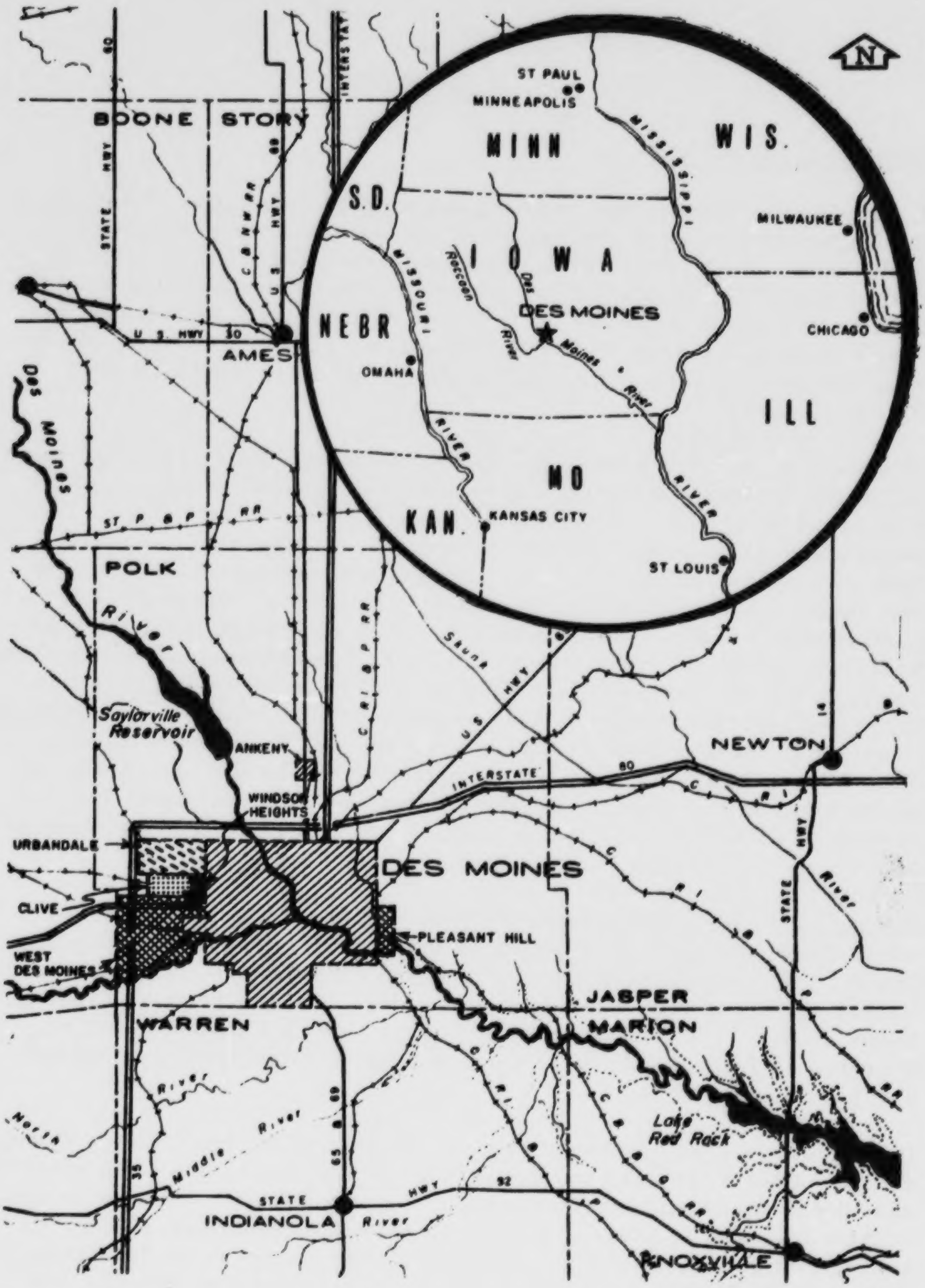

N
ST PAUL
MINNEAPOLIS
MINN
WIS.
MISSISSIPPI
S.D.
MILWAUKEE
IOWA
MISSOURI
Raccoon River
Des Moines River
DES MOINES
CHICAGO
NEBR
OMAHA
ILL
RIVER
MO
KAN.
KANSAS CITY
RIVER
ST LOUIS
BOONE
STORY
STATE HWY 60
C & N W RR
U S HWY 69
INTERSTATE
AMES
U S HWY 30
Des Moines
ST P & P RR
POLK
River
Saylorville Reservoir
ANKENY
C R I & P RR
Skunk
U S HWY
WINDSOR HEIGHTS
INTERSTATE
80
C R I
NEWTON
14
STATE HWY
River
URBANDALE
DES MOINES
CLIVE
PLEASANT HILL
WEST DES MOINES
JASPER
MARION
WARREN
River
North
Middle
River
65 & 69
35
C R I & P
C B & Q RR
Lake Red Rock
STATE HWY 92
INDIANOLA
River
KNOXVILLE

VICINITY MAP

FIGURE I

to late March, has a normal mean temperature of less than 40 degrees. The summer season, which lasts about 21 weeks from early May to the beginning of October, has a mean daily temperature of 60 degrees. The spring growing season and the fall harvest season range between 40 degrees to 59 degrees, each lasting about 6 weeks.

Precipitation averages about 31.5 inches annually, with a minimum of 17.1 inches in 1956 and a maximum of 56.8 inches in 1881. The monthly variation is even more remarkable, from the very dry month of October 1952, with only .03 inches precipitation to the very wet month of June 1881, with 15.79 inches of precipitation. Average precipitation for the winter season is about 6 inches, or approximately 20 percent of the annual amount. Beginning with the spring growing season, the frequency and intensity of precipitation increases markedly to a maximum in June. The total growing season averages about 25 inches, or approximately 80 percent of the annual total. The average for the 21 weeks of summer is about 18 inches. Record mean monthly precipitation is given in Table 1.

TABLE 1

MEAN MONTHLY PRECIPITATION

January	1.14	July	3.33
February	1.13	August	3.54
March	1.92	September	3.38
April	2.78	October	2.33
May	4.28	November	1.56
June	4.72	December	1.16

Annual 31.27

The Raccoon River is the principle source of water for the area. The Des Moines Water Works supplies all of the developed areas of the City of Des Moines and wholesales water to the communities of Windsor Heights, Clive, Urbandale, Pleasant Hill and a number of private residential and industrial users in the area. The remaining cities and towns in the Study Area operate their own water supply system, drawing principally from the Jordan Formation at depths of 2,500 feet and greater. Shallow wells in the sand and gravels of the North River are also used for municipal water supply.

Local terrain features have lent themselves to cooperation in the planning and utilization of sewage collection systems and treatment facilities for many years. Since the original City of Des Moines Sewage Treatment Plant was placed into service in 1939, it has treated all of the organic waste from Des Moines and the City of West Des Moines. The Des Moines system also presently receives waste from the cities of Clive, Windsor Heights, and a portion of Urbandale, which are all adjacent to Des Moines on the west.

STUDY AREA

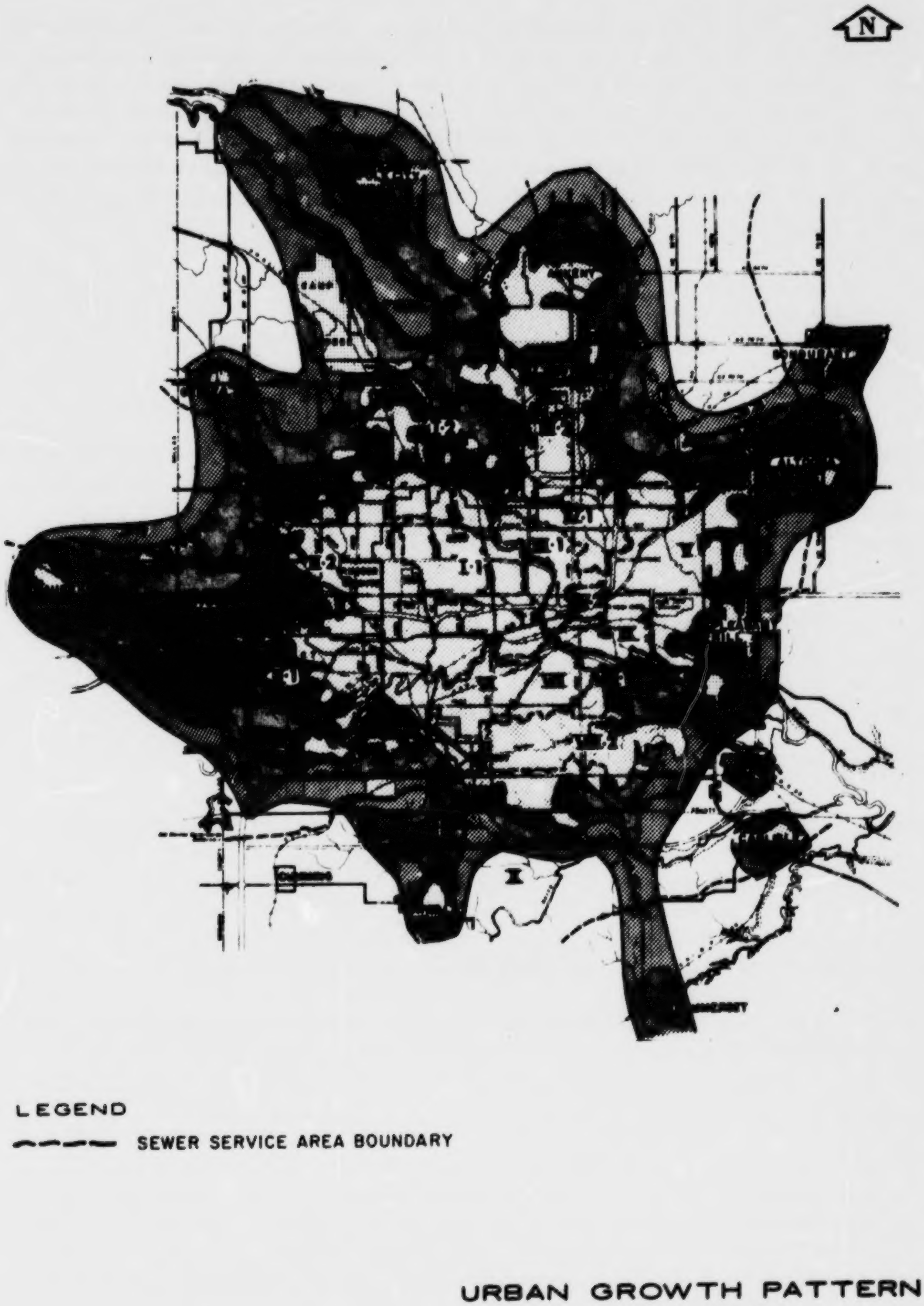
The basic study area is the urban area of Des Moines and the surrounding communities. Field data collected for the study was generally obtained from within the present urban boundaries. In the river sampling program, monitoring was conducted above Des Moines as well to obtain basic river quality data. Present and future requirements were based on conditions within the projected urbanized areas shown on Figure 2.

Specific studies were also conducted in selected areas. While the rainfall monitoring network was designed to provide coverage of all the urban area, rainfall-runoff analysis were limited to five well-defined watersheds. A detached analysis of combined sewer separation was conducted for an area of approximately 1800 acres in the west-central part of the City. The areas for such specific studies are described in detail in subsequent sections.

GROWTH PATTERNS

To evaluate the influence of urban growth on wastewater collection and treatment requirements, a study of the past, present and future population and industrial growth is necessary. Systems designed to carry storm and sanitary flow in combined sewers must receive particular attention, since contributions to existing trunk and interceptor sewers are compounded by growth in upper reaches of the basin. Even where tributary growth areas have separate storm and sanitary systems, the waste flowing into the combined system carries an increasing pollutional load, which will be subject to overflow during wet weather periods.

Growth for the Study Area was projected for a 50 year period. A 20 year period was considered for wastewater treatment requirements since it is generally both unsound planning to construct facilities designed beyond this period and uneconomical to pay interest on an investment which will not be brought into efficient use within the normal 20 year financing period. Interim facilities may be designed for shorter periods, but should be considered in a long range plan. Since the life of a sewer can reasonably be expected to be 50 to 75 years, sewer design, whether storm or sanitary, usually takes into consideration the ultimate development of a drainage basin insofar as practical. Where extensive drainage basins are involved, such as in the major rivers and streams, the 50 year growth pattern is determined and the sewers designed accordingly.



LEGEND

—— SEWER SERVICE AREA BOUNDARY

URBAN GROWTH PATTERN
1970 - 2020

FIGURE 2

Figure 2, entitled "Urban Growth Patterns" depicts the anticipated areas of growth as foreseen by the Central Iowa Regional Planning Commission. Population projections for this study considered the entire growth within the 2020 urban boundary, with the exception of that area south of the North River basin. The growth Pattern overlays the major sanitary sewer watershed boundaries so the extent of growth in each basin can be envisioned. Sewer service areas designated by Roman numerals in Figure 2 are titled as follows:

I - West Side Interceptor
II - Southwest Outfall
III - East Side Interceptor
IV - East 18th Street Trunk
V - 4-Mile Trunk
VI - Southern Hills Trunk
VII - South Side Trunk
VIII - Bloomfield System
IX - Main Outfall
X - North River Basin

Strong growth to the west is anticipated in the seventies, as well as settlement of the unincorporated areas north of Des Moines including the corridor running to the City of Ankeny. Between 1980 and 2000 the growth should swing strongly north and northwest as fingers running generally along the ridge lines to the existing towns north and west of Des Moines. The Saylorville Reservoir, now under construction, will act as a magnet for future growth to continue strongly to the northwest through 2020. It is of interest to note that, with the exception of the southerly growth into the North River Basin, nearly all growth is within and tributary to populated watersheds.

POPULATION PROJECTION

Numerous sources of information were investigated to determine an appropriate growth estimate for the Des Moines urban area. Past and present populations were determined from census information. Population estimates from recent engineering reports were reviewed and population estimates and anticipated growth patterns to the year 2020 were furnished by the Central Iowa Regional Planning Commission.

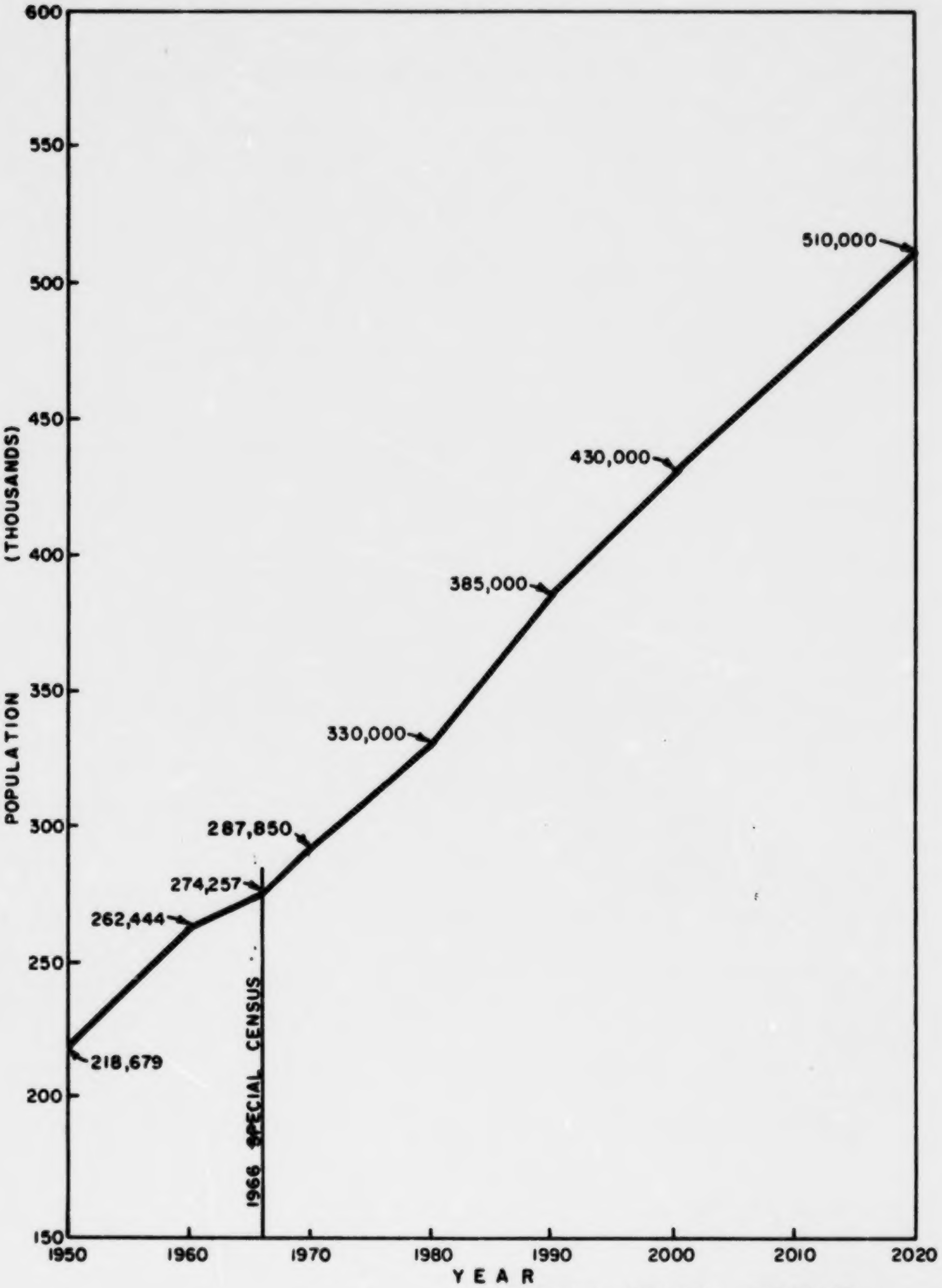
Table 2 lists the cities, towns and parts of counties included in the 2020 growth area and gives the populations for each from recent census reports. The estimated future population for the Des Moines urban area is also shown graphically in Figure 3. The 1970 population is estimated to be 287,850, increasing to 330,000 by 1980, 385,000 by 1990, and 510,000 by 2020. This estimate is very close to that prepared by the firm of Howard, Needles, Tammen and Bergendoff in a planning report for the Central Iowa Regional Planning Commission. The CIRPC report, which was done concurrently with this study, put the 1970 urban population at 280,500, with 241,500 in 1980, 426,000 in 2000 and 511,500 in 2020.

TABLE 2

URBAN AREA POPULATIONS

	1950	1960	1965-1966 Special Census	1970
Altoona	763	1,458	2,424	2,854
Ankeny	1,229	2,964	5,910	9,151
Bondurant	328	389	500*	462
Carlisle	903	1,317	1,930	2,246
Clive	--	752	1,735	3,005
Des Moines	177,965	208,982	206,739	200,587
Grimes	582	697	800*	834
Norwalk	435	1,328	1,630	1,745
Pleasant Hill	--	397	1,006	1,535
Polk City	336	567	700*	715
Urbandale	1,777	5,821	10,310	14,434
Waukee	431	687	480	1,577
West Des Moines	5,615	11,949	13,720	16,441
Windsor Heights	1,414	4,715	6,409	6,303
County Areas (Polk, Dallas, Warren)	26,831*	20,421*	19,177*	25,961*
Total	218,679	262,444	274,257	287,850

*Estimated Populations



POPULATION TREND
DES MOINES URBAN AREA

FIGURE 3

Combined sewers exist only in the older sections of the city, which are generally developed to near saturation density. For this reason, only small increases in population are predicted for combined sewer areas. The areas served by combined sewers are listed in Table 3 with present and estimated future populations. These service areas can be located on Figure 2.

TABLE 3

COMBINED SEWER AREAS POPULATIONS

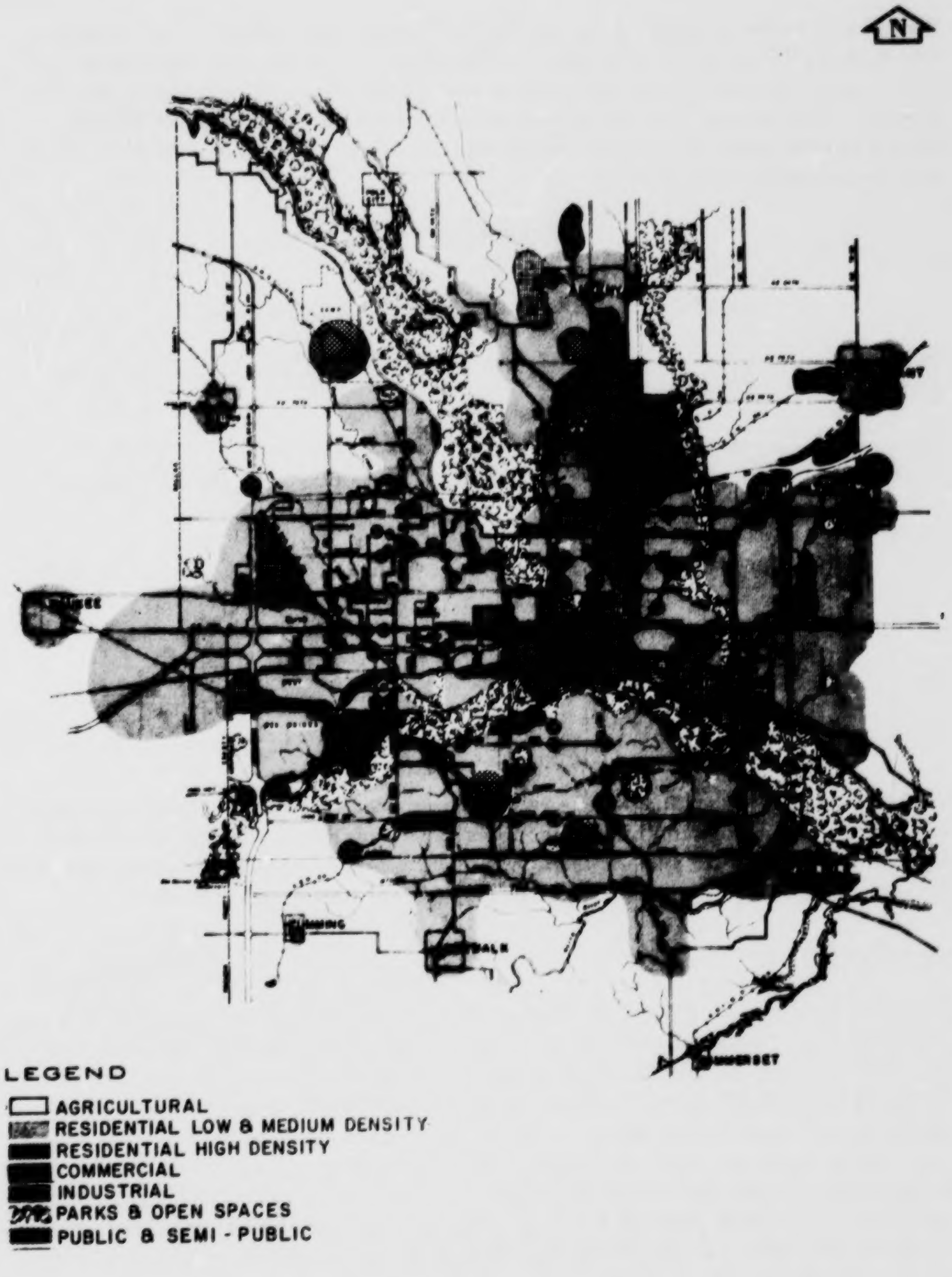
Service Area No.	Description	Estimated Population 1969-1970	1990	2020
I-1	West Side Intercepter, Central City	79,000	81,000	84,000
III-1	East Side Intercepter, Central City	16,400	17,500	19,000
IV-2	East 18th Street Trunk	7,000	7,100	7,300
VI	South Side Trunk	15,300	18,000	20,000
	Total	117,700	123,600	130,300

Although some portions of these service areas are not served by combined sewers, flow in the trunk collector sewers is combined sewage. Similarly, it should be noted that all wastewaters, except from the Four-Mile Trunk, is conveyed by the main outfall, a combined sewer.

LAND USE

Future land use is shown in Figure 4. Of particular interest is the industrial belt in the northeast sector running along Interstate 235 from the City of Ankeny southward to the southeast bottoms on the left bank of the Des Moines River. Nearly all of the existing industries which contribute heavy pollutional loads to the waste treatment facilities are located in this corridor and fall mainly within the East 18th Street sanitary sewer drainage basin (Area IV - 1 and 2 in Figure 2). This industrial belt is significant with respect to combined sewer problems. The City of Des Moines should strongly encourage wet process industries to locate within this corridor.

Commercial and high density residential areas are located to a large



N
SOMERSET
LEGEND
AGRICULTURAL
RESIDENTIAL LOW & MEDIUM DENSITY
RESIDENTIAL HIGH DENSITY
COMMERCIAL
INDUSTRIAL
PARKS & OPEN SPACES
PUBLIC & SEMI - PUBLIC

FUTURE LAND USE

FIGURE 4

extent near the center of the city on the east and west side of the Des Moines River, an area served primary by combined sewer systems.

Also of interest is the designation of parks and open spaces along the Des Moines and Raccoon Rivers. Above the city on the Des Moines River, the Saylorville Reservoir is currently under construction. Parks and open spaces are also designated downstream of the city where the flood pool of the recently completed Red Rock Reservoir reaches to within the city limits.

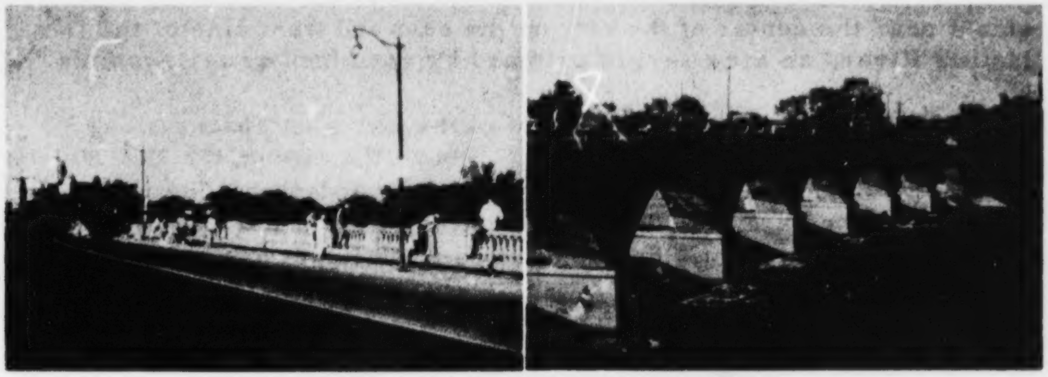
DES MOINES AND RACCOON RIVERS: USAGE-POTENTIAL

Both rivers are subjected to extensive usage within the metropolitan area. The Raccoon River is the primary source of water supply for the Des Moines Water Works. A recent utilities inventory prepared for the Central Iowa Regional Planning Commission indicates the existing supply is adequate for present and immediate future demand. However, long range demand may require an additional source and the utilities inventory pointed to the Des Moines River, specifically Saylorville Dam, as the likely source. At present, because of its use as a water supply and because of its lower flow and shallower depths, the Raccoon River is not used extensively for water sports recreation.

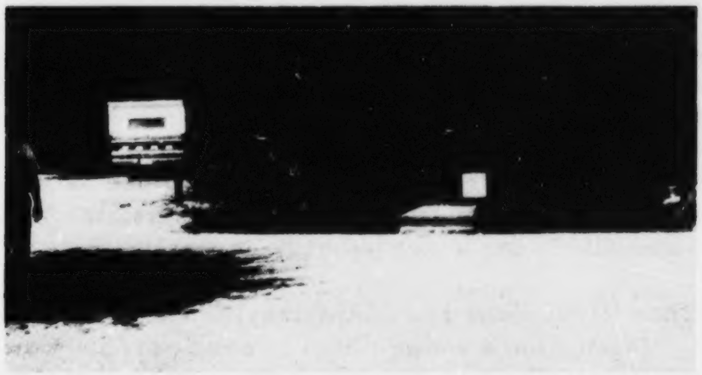
The Des Moines River does receive extensive water sports and recreational usage. During open water there is considerable fishing pressure throughout its course through the City. During the summer, a low head dam at Center Street maintains sufficient depth in the section upstream to Euclid Avenue to provide excellent boating opportunity. There are two private marinas and several public launching areas in this stretch of the river. Figure 5 illustrates some of the recreational uses.

It is highly improbable that recreational use of the Des Moines River will decrease, even with the recent completion of Red Rock Dam downstream from the city and completion in 1975 of Saylorville Dam immediately upstream. If anything, recreational usage will probably increase, due partly to the improved quality of the river which should result from the Saylorville impoundment.

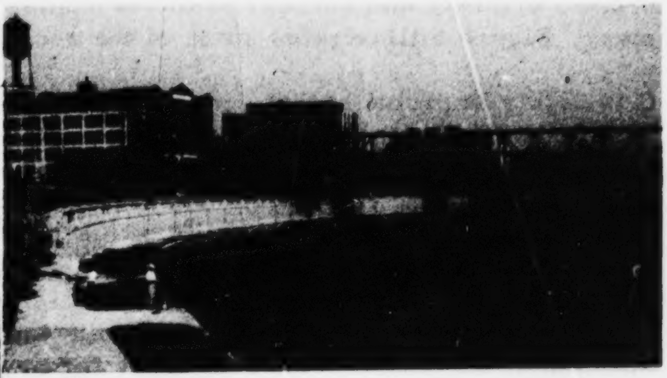
The location of two major impoundments immediately above and below the metropolitan area, and especially the Saylorville impoundment upstream, provides an excellent opportunity for the City of Des Moines to have, within its confines, a water sports and recreation area of significant social and economic benefit. The multiple advantages of having a clean clear river through the heart of the city and metropolitan area are almost innumerable. Land use planning appears to anticipate this, since Figure 4 shows parks and open spaces planned for most of the river frontage with exception of the downtown area.



FISHING AT SCOTT STREET



BOAT RAMP NEAR STATION O-2



FISHING FROM TOP OF WEST SIDE STORM BOX

WATER SPORTS RECREATION WITHIN DES MOINES

FIGURE 5

SECTION V

FIELD OPERATIONS AND PROCEDURES

GENERAL INFORMATION

This section describes the general concept and execution of the data collection and analyses. To minimize duplication of written material and cross-referencing, all monitoring and sampling stations are described in Section VI, with detailed descriptions given in Appendix A. Presentation and discussion of sample and flow data is concentrated in Section VI.

Field operations were conducted from an office established near the center of the project area at East 1st and Locust Streets. This central location was very advantageous for dispatching personnel quickly to sample points during runoff. Office facilities were such that all field equipment could be easily moved in and out as necessary for service, cleaning and repair. Two carryall type vehicles were used for transporting tools and equipment and a station wagon was available for lighter uses.

Field operations were in 8 general areas of endeavor: (1) install and service monitoring and sampling equipment; (2) conduct scheduled sample programs such as collection of dry-weather sanitary and river quality data; (3) spontaneous flow measurement and sampling during runoff periods; (4) current meter flow measurements to establish flow curves and hydraulic coefficients; (5) equipment maintenance; (6) preliminary workup of flow and laboratory data during slack periods between runoffs; (7) surveying and collection of topographic data; (8) liaison with local and regional governmental agencies for program coordination and collection of available related data.

During normal operation, several of the above activities could be conducted simultaneously. During rainfall, however, all effort was directed toward measurement and sampling of runoff. Following the sample periods, it was necessary to spend a considerable amount of time on sample disposition and clean-up of equipment. Routine operation and servicing of equipment in the field was accomplished by maintaining a schedule of twice weekly visits to each station. This schedule, once implemented and ironed out, was satisfactory for continuous operation of the equipment.

MONITORING AND SAMPLING EQUIPMENT

Water Level Monitoring Equipment

Three types of devices were used to detect overflow and/or record water levels. They were: (1) stick gauges painted with a water soluble paint; (2) drum-type 1 or 7 day float operated recorders; and (3) compressed air 1 or 7 day bubbler type recorders.

Painted stick gauges were used at the outset of the project, while mechanical equipment was on order, to gauge frequency of overflow at numerous points. The information obtained was helpful in determining which overflows should be continuously monitored. The stick gauges were simply 1" by 2" boards stapled in place with a ram-set or wedged into sewer outlets. High water levels could usually be determined quite accurately by this method, since the water soluable paint would be washed off to the high water level.

The float operated recorders were Leopold and Stevens Type F Recorder, Model 68, with interchangeable gears to enable 24-hour or 7-day operation. These units were used mainly for temporary setups such as the dry weather sanitary sampling. The units are light-weight, portable and perform very well in temporary installations. Figure 6 shows installations of the float operated recorders at three sampling stations. Where turbulent flows are encountered, the float must be protected with a float tube. Six-inch fibre pipe was found to work quite well for this purpose.

The compressed-air bubbler recorders were designed and fabricated as a part of this project. The bubbler units were used for all semi-permanent monitoring points. The unit is shown on Figure 7. Also shown are installations at Sampling Stations 0-6 and 0-11. The basic component parts of the unit were: (1) Foxboro Model 12R 12-inch circular case pressure recorder, (2) Foxboro Purge Rotometer D-105-NX for air feed control, (3) Foxboro Type 67 supply regulator with 2-inch MH pressure gauge, (4) Cornelius CO_2 Type Regulator, Model 28433-1 with inlet and outlet pressure gages, for initial pressure reduction from air cylinder, (5) 20 lb. compressed air cylinder, and (6) vandal-proof steel enclosure. Shown in the foreground at the bottom of the bubbler in Figure 7 is an air valve which was used unsuccessfully to attempt to automate sampler startup at a preset water level. This problem is discussed with the sampling equipment.

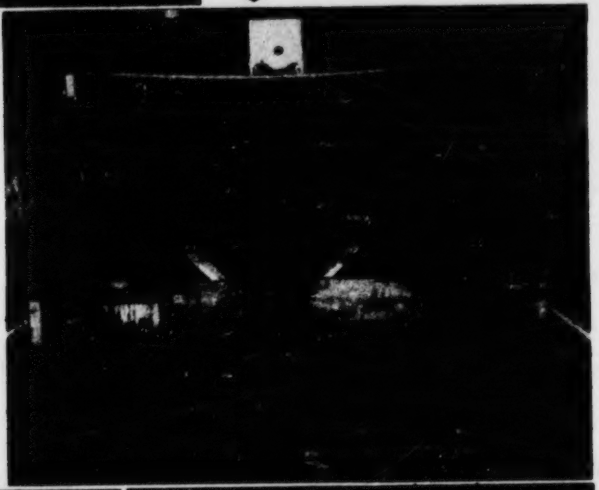
The bubbler unit had several distinct advantages which dictated its selection for this project. Some of them were:

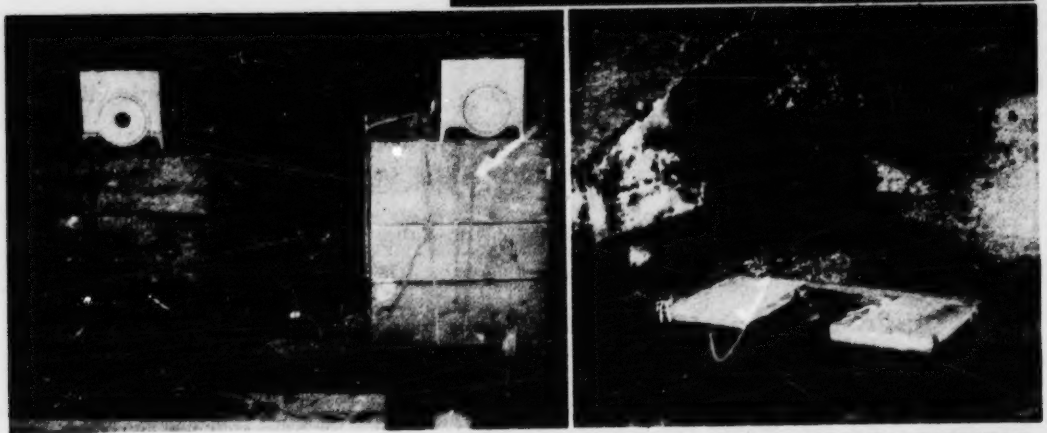
1. The unit was portable and self-sufficient. External power was not required.

2. Routine maintenance was minimal, consisting mainly of winding the clock and changing the chart on a weekly basis and changing the air bottle at 1 to 4 week intervals, depending on the station.

5'X13' INGERSOL RUN BOX AT D-1A

IN 5'X 6' UNIVERSITY AVE. BOX AT S-2





SURCHARGING PROMPTED PREMATURE REMOVAL OF RECORDERS AT SIPHON OUTLET CHANNELS IN D-1

TYPICAL WIER & RECORDER INSTALLATIONS

FIGURE 6

BUBBLER UNIT

STATION O-11
BUBBLER AND SAMPLER
INSTALLED AT END OF BOX
SEWER

STATION O-6
BUBBLER INSTALLED IN
MANHOLE IN DRIVEWAY

TYPICAL BUBBLER INSTALLATIONS

FIGURE 7

3. The unit could be installed at considerable distance from the actual monitoring point. At several stations, this distance was 50 to 75 feet. In these and all cases, the plastic air line was enclosed in aluminum electrical conduit for protection against debris and vandals.

4. When secured in place with ram-set bolts and the cover padlocked, the unit was virtually vandalproof.

5. The recording head could be used with either 24-hour or 7-day charts and required only a simple screw adjustment to change from one to another.

6. There were no moving parts in the monitored flow and no significant obstruction in the flow pattern. Also, the operation was not affected by freezing or ice cover.

7. The basic principal of operation is widely used in water and wastewater operations and generally familiar to operating personnel.

8. The fabricated unit, including assembly, was relatively inexpensive, about $350 each.

The compressed air cylinders were 20 lb. cylinders furnished and filled for a fee of $2.50 per tank by a local supplier of moisture-free air for skin divers. As noted earlier, a full tank, 2,000 psi, would last from 1 to 4 weeks. Most stations carried 2 to 3 tanks per month.

Automatic Sampling Equipment

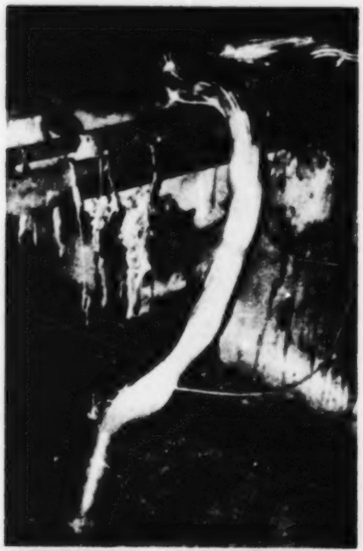
The sampler selected for this project was the Serco Automatic Sampler, Model NW3-8. This unit is an automatic, timer actuated, vacuum sampler which will collect 24 consecutive individual samples at equal intervals. For this project, 6-hour and 24-hour clocks were used, thus providing the flexibility of selecting 15 minute or 60 minute sample intervals. The quarter-hour samples were especially advantageous for flash runoffs.

Figure 8 shows several photos of the sampler and one series of samples. The sample bottles hold about 500 ml. Usually 300 to 350 ml., depending on suction lift, would be obtained. Where possible, suction lift was minimized to 5 to 8 feet to obtain the maximum sample. On occasion, however, samples were successfully collected with lifts of 10 to 11 feet.

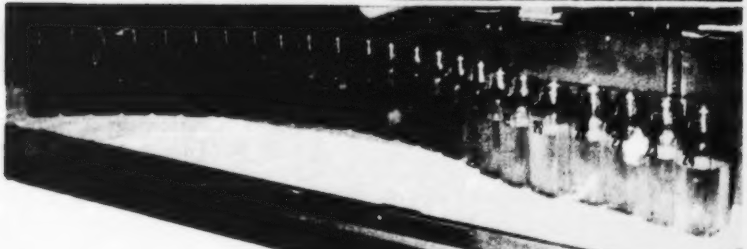
SAMPLER –
WITH SAMPLES

CLEANING SAMPLER
BY BACKFLUSHING

SAMPLER HEAD IN
PLACE AT D - 1



COMPLETED
SAMPLE RUN
FROM D - 2



AUTOMATIC SAMPLING EQUIPMENT

FIGURE 8

An insulated metal case houses the sampler in the field. The entire unit, including case, will just pass through a 21-inch manhole opening. For some sample points, it was necessary to suspend the sampler in a manhole in the roadway. This was done by using a belt harness suspended from a reinforced recessed hanger bar which permitted replacement of the manhole cover following placement of the sampler. For stations where the sampler could sit on top of or along side the sewer or stream, a 55 gallon drum was used to cover the unit and minimize vandalism. This was especially necessary when charged samplers were left in the field for extended periods between rainfall. "L"-shaped strap iron was welded to the base of the drum and slotted to permit padlocking the drum to short sections of chain secured to the support structure.

The particular sampler used has its advantages and some disadvantages. Some of the good points are:

1. The sampler, when primed, is self-sufficient and therefore does not require an external source of power.

2. Because of "1" above, the sampler can be used in remote locations, such as inside large sewers.

3. The sampler case is well insulated and when properly iced will adequately cool samples for at least 24 hours.

4. Collection of individual samples rather than one composite sample gives the engineer the flexibility of analyzing individual samples, a composite sample, or both.

5. "4" above also permits visual observation of each sample.

6. The sampler unit is portable and is light enough that one man can, if necessary, handle it. Two men make the job considerably easier, however.

Some of the disadvantages are:

1. Some features of the sampler, in particular the tripping mechanism and the sampler head, are of relatively light construction for the type of use required for this particular project. Repairs were required on several occasions during the 11 months of usage.

2. The clocks provided are not adequately moisture-proof and late in the project were starting to malfunction.

3. The area beneath the sampler handle bar is extremely limited. It was very difficult to set or start a charged sampler without tripping one or more of the bottles. This problem is magnified in cold weather.

4. The trip arm is rotated by a shaft from the clock and slippage is prevented by friction from a knurl headed bolt which must be completely tight or the trip arm will slip on the shaft. It is sometimes difficult to determine when the bolt is tight and several sampling opportunities were missed because of this problem. A more positive means of securing the trip arm would be desirable.

5. The sampler head and the 1/4" suction tubes are subject to clogging by paper, leaves, worms, etc.

One original goal in automatic sampling procedure was never realized. That was perfection of a device to actuate the sampling sequence automatically at some predetermined depth of flow. A device for that purpose was furnished by the sampler supplier but could not be made to work within the limitations of the air supply. That device depended on increasing submergence to release a vacuum which would retract a plunger retaining the trip arm, thus starting the sampling sequence. The problem was in the vacuum release step. For the flow situations in this project, submergence was gradual rather than sudden; thus the vacuum seeped away rather than suddenly retracting the plunger.

Considerable time was spent trying to modify this operation to use pressure from the bubbler unit, rather than vacuum, to lift or retract the plunger. A double action diaphragm valve (from the heating system of a junked automobile), mounted beneath the handle of the sampler and connected to the air control valve shown in Figure 7, was to retract the plunger at a preset pressure. Although bench tests were successful, the device did not prove reliable for field operations. Moisture, grime, ants, etc. soon fouled the needle-type air valve and several mid-night sample opportunities were missed. For the balance of the project, the samplers were started manually.

FLOW MEASUREMENT-HYDRAULICS

Hydraulic studies and analyses were an integral, and sometimes unsolvable, part of the field operation. Measured flows ranged from 1 MGD dry weather sanitary flow to greater than 200 CFS open channel storm runoff. Various schemes were used to determine discharge, as described hereafter.

Weirs

Weirs were used whenever possible and practical. All dry weather sanitary flows were measured by this procedure. Figures 6, 9 and 10 show weir installations at D-1A, D-2, S-2 and S-3. Basic weir installations were all of the same general nature. Wooden bulkheads were premeasured and cut and then assembled inside the sewer or channel. The steel weir plate was then bolted to the bulkhead. Joints were caulked and lapped and burlap was stuffed around the perimeter to minimize leakage. In the larger sewers, downstream bracing was necessary to retain the bulkhead in place. Water level (head) was recorded with either the float type or bubbler recorders described previously. The following weir formulae were used to compute discharge:

90° V-Notch Weir	$Q = 2.54H^{5/2}$
Rectangular Suppressed Weir	$Q = 3.33\ LH^{3/2}$
Rectangular Contracted Weir	$Q = 2.22H^{3/2}\ (L-0.2H)$

Where H = Head on weir in feet
L = Length of weir in feet
Q = Flow in CFS

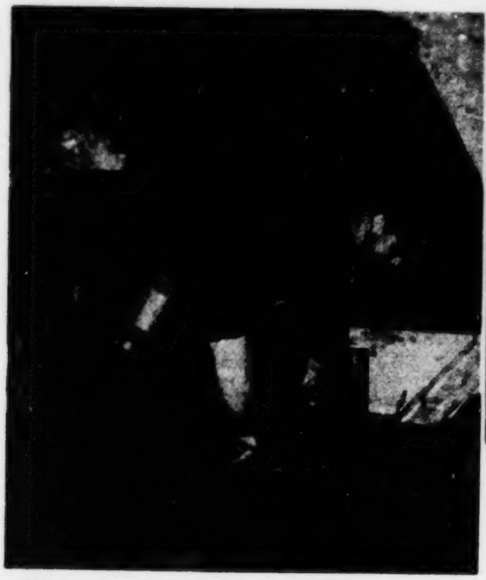
Stage - Discharge Relationship

Stations 0-2 and 0-8 were located in drainageways which carried very high volumes of water. The only feasible way to determine discharge for these points was to establish a stage-discharge curve by frequent area-velocity measurements utilizing a rated current meter. Fortunately, there were permanent natural or artificial pool controls in the immediate vicinity of both stations. This fortunate circumstance precluded the need for shifts of the curve to correct for control movement. Figure 10 shows the stations and controls.

Shifts were not eliminated entirely, however. At Station 0-2 semipermanent plus and minus shifts were necessary to compensate for several changes of the bubbler zero datum.

Hydraulic Gradient Computations

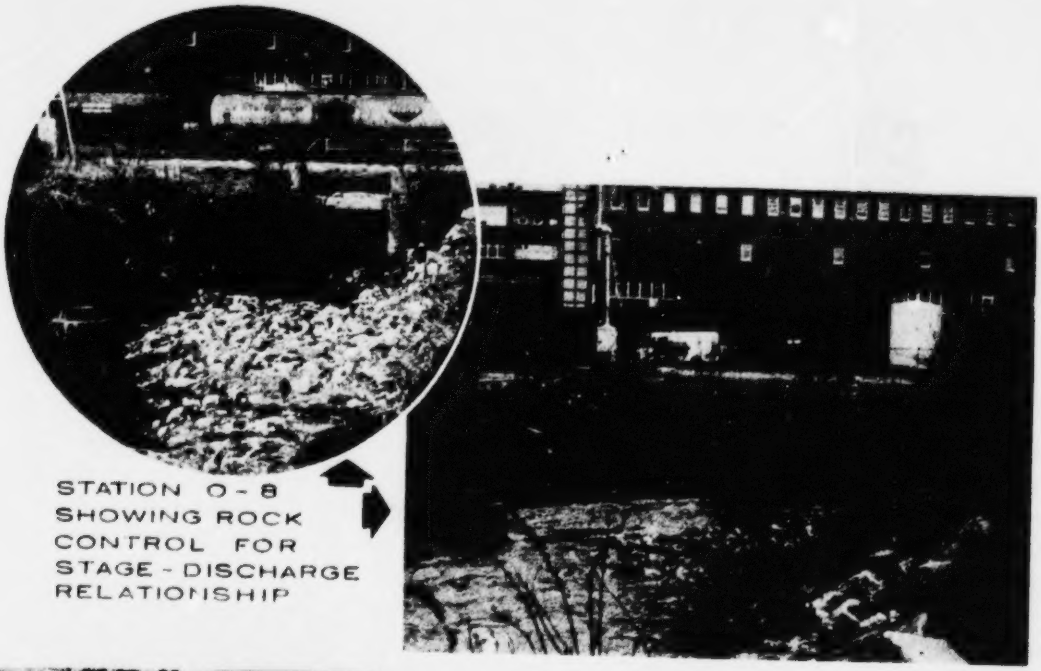
Flows in conduits can be computed from the Manning Formula:



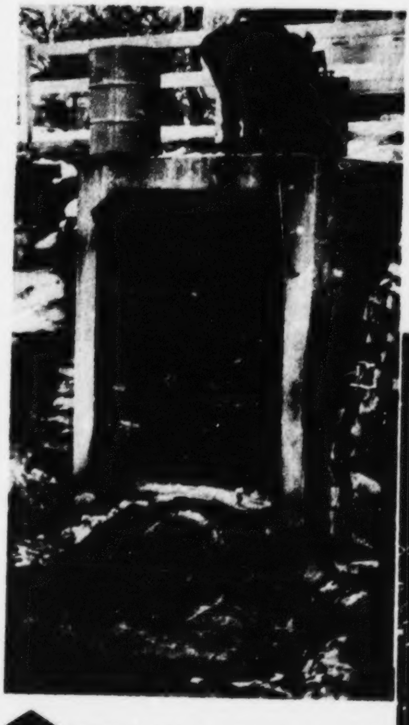
WEIR CONSTRUCTION IN 78"
SOUTHWEST OUTFALL SEWER
AT STATION D-2

WEIR CONSTRUCTION

FIGURE 9



STATION O - 8
SHOWING ROCK
CONTROL FOR
STAGE - DISCHARGE
RELATIONSHIP



STATION S - 3
WEIR, BUBBLER AND
SAMPLER SETUP

STATION O - 2
SHOWING CONCRETE
SEWER ENCASEMENT
WHICH SERVED AS CONTROL

DISCHARGE MEASURING FACILITIES

FIGURE 10

$$Q = \frac{1.486}{n}AR^{2/3}S^{1/2}$$

Where Q = discharge in CFS

A = Cross-sectional area in ft.^{2}

n = Coefficient of roughness

R = Hydraulic radius of flow section in ft.^{2}/ft.

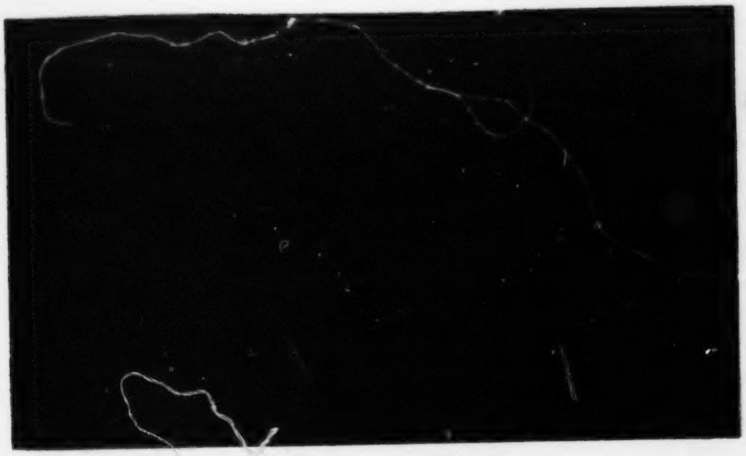
S = Slope of the hydraulic gradient in ft./ft.

Station 0-6 flows were computed by this formula. The input data was obtained by establishing bubbler stations 0-5 and 0-6 which were on the same sewer at a known distance apart. The bubblers were referenced to City datum so that the true water surface elevation could be determined. Using this data, plus sewer dimensions, invert elevations and sludge depth, "A", "R" and "S" could be computed. The coefficient of roughness "n" was determined by measuring actual flow with the current meter and computing "n." For this 60-inch brick sewer, "n" was determined to be 0.018. Using the above information, flow tables were prepared for combinations of the three variables - sludge depth, total depth and hydraulic gradient (slope).

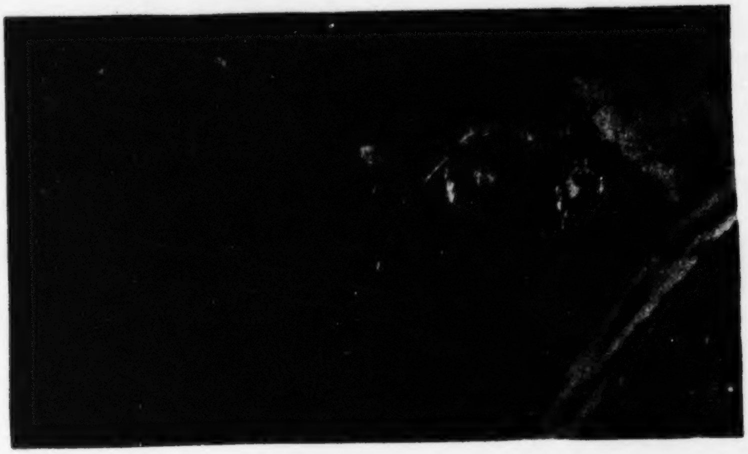
Hydraulic gradient versus flow relationships were also attemped at siphon locations D-1 and D-5 and for the West Side Storm Box 0-7. The results were not reliable. Variable indeterminate restrictions in the lines between the monitoring points, and intermittent backup from downstream surcharging, introduced too many variables. Figure 11 shows some of the conditions which adversely affected hydraulic gradient computations for the West Side Storm Box.

Current Meter Measurements

Current meter flow measurements were used extensively to spot check flows throughout the sewer system in an attempt to locate the sources of excessive extraneous flow. The procedure used was to measure depth of flow, depth of sludge, and average mid-point velocity and from that data compute the flow. The accuracy of this procedure was checked several times by measuring flows at locations upstream from weir installations. Flows by current meter measurement were usually 5 to 7 percent higher than those determined by weirs. Figure 12 shows the current meter equipment being used for a sewer flow measurement.



MUD ACCUMULATION AT BIRDS RUN OUTLET, WEST SIDE STORM BOX



LEAKS ALONG RIVER SIDE OF WEST SIDE STORM BOX

DEBRIS IN 6'X13' WEST SIDE STORM BOX BELOW BIRDS RUN OUTLET

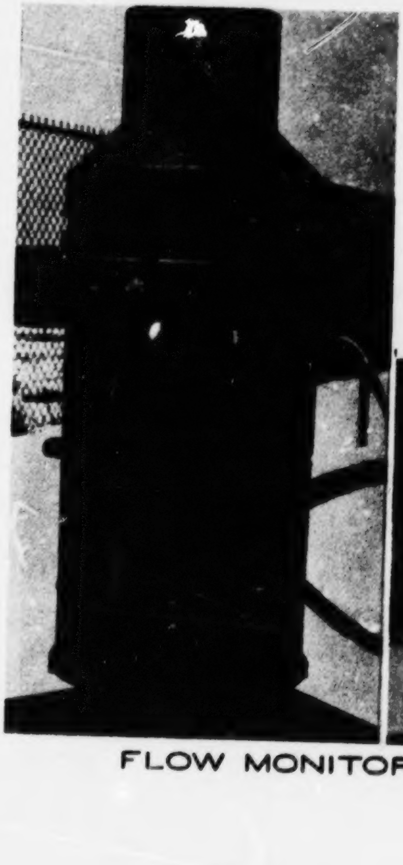
WEST SIDE STORM BOX

FIGURE 11

MEASURING SEWER FLOW WITH CURRENT METER

RIVER SAMPLING AT R-9

RAIN GAUGE INSTALLATION AT KSO (R.G. NO. 3) WAS UNIQUE



FLOW MONITORING & RAIN GAUGE INSTALLATION

FIGURE 12

RAINFALL MONITORING EQUIPMENT

Recording rain gauges were placed at six locations within the study area to supplement two existing rain gauges maintained by the U. S. Weather Bureau. Gauges were the weighing-type gauge, with 48-hour chart rotation and 8-day chart drive clocks. Extra chart drums were ordered after a short period of operation so that charts could be changed in the office, which proved to be considerably easier than changing in the field. Except for one gauge located at KSO Radio Broadcasting Company, temporary platforms were constructed of steel fence posts and scrap formica-covered plywood bases. At the KSO site, a fenced swimming pool which is no longer in use provided a secure area for the gauge. The diving board platform served as an excellent base for the rain gauge, as shown in Figure 12.

All of the sites were fenced areas and no vandalism problems were encountered. Arrangements were made with each owner to obtain a key to the premises or to maintain the gauges when the premises were open.

Gauges were checked and charts changed routinely on a twice per week basis, generally on Monday and Friday. This provided 1-1/2 to 2 rotations of the chart, but did not interfere with reading the chart. Hourly precipitation was tabulated for each of the six supplemental gauging stations in the same format as used by the Weather Bureau. Two items varied from standard Weather Bureau practice. One was that trace amounts were not recorded as such, but an 0.01 inch depth in the hour during which the accumulation reached 0.01 inch above the previous reading. Secondly, due to need to correlate rainfall and runoff records, daylight savings time was used when applicable as opposed to the Weather Bureau practice of using standard time throughout the year.

SAMPLE COLLECTION AND ANALYSIS

General sampling operations and sample analysis by the various laboratories are discussed herein. Dry weather sanitary samples were all 24-hour composite samples for 2 to 3 days consecutively. Wet weather sanitary sampling, combined overflow sampling and storm water discharge sampling were analyzed as either grab samples or composites of varying duration. The determination of when to composite samples or when to analyze grab samples was generally based on the percent of the storm actually sampled and the desire to obtain peak flow BOD's, DO's or other constituents. Usually when samples were collected for only a part of the runoff, grab samples were analyzed for significant points in the flow curve.

Collection and Preservation

Sanitary sewage, combined overflow and storm water discharge samples were usually collected with the automatic samplers. The major exception was the initial dry weather sanitary sampling which was hand collected prior to receipt of the samplers. River samples were collected by hand, usually from bridges. The small creeks were waded.

Samples for sanitary analysis (BOD, solids, nitrogens and phosphates) were 1000 to 3000 ml and were iced to retard degradation. Samples for coliform analysis were collected in sterilized bottles provided by the laboratory and were taken with the use of a wire basket fabricated for the particular bottles used. The basket could be suspended from a bridge with a rope and precluded contamination of the bottle during the sampling process. Samples for plankton were collected in a one liter bottle which contained a preservative when received from the laboratory. Samples for pesticide were collected in glass quart jars which had undergone special cleaning procedures in the laboratory.

Dissolved oxygen samples were collected in 300 ml DO bottles with the use of a DO dunker. The samples were "fixed" or set immediately and then titrated upon return to the office. Samples for all analyses not discussed above were collected in gallon jugs with no special handling or preservation. In all cases samples were delivered to the laboratories as soon as possible following collection. Figure 12 shows a member of the field crew collecting a river sample at Station R-9.

Laboratories

All sample analyses, except the temperature, pH and dissolved oxygen tests done by the field crew, were contracted to three laboratories. The laboratories and the tests performed by each were as follows:

1. University of Iowa, State Hygienic Laboratory at Iowa City - Pesticide analyses (DDT, DDE, Dieldren)
2. City of Des Moines, Department of Public Health Laboratory - Coliform analysis, total and fecal
3. Iowa State University, Engineering Research Institute Laboratory

	Abbv. used in this report
Biochemical Oxygen Demand	B. O. D.
Chemical Oxygen Demand	C. O. D.

	Abbv. used in this report
Ammonia Nitrogen	$NH_3.N$
Nitrite Nitrogen	$NO_2.N$
Nitrate Nitrogen	$NO_3.N$
Alkalinity	
Hardness	
Color	
Turbidity	
Sulfates	
Total Phosphate	$T.PO_4$
Soluble Phosphate	$O.PO_4$
Total Solids-Volatile and Non-Volatile	
Suspended Solids-Volatile and Non-Volatile	
Dissolved Solids-Volatile and Non-Volatile	
Calcium	
Sodium	
Chlorides	Cl
Chlorine Demand-1 hour	
Plankton	
Chromium	Cr
Cyanide	

All analyses were performed in accordance with the procedures outlined in Standard Methods for the Examination of Water and Wastewater, 12th Edition,(2) except for the following major exceptions.

1. Use of 1/40 N. Phenylarsine oxide instead of sodium thiosulfate for DO and BOD titrations.

2. Use of glass fibre mats instead of asbestos mats for suspended solids filtration. This modification necessitated ignition of the residue at 580° C, instead of 600° C, to preclude ignition of the mat.

The above modifications were accepted after the laboratory had demonstrated satisfactory accuracy with their usage. As a matter of interest, these modifications are also used for the Saylorville Dam Preimpoundment Study being conducted by the I. S. U. Engineering Research Institute, an outside source of data for the analyses in Section 6 of this project.

SECTION VI

SEWAGE, OVERFLOW AND STORM WATER DISCHARGE DATA

GENERAL

This section describes the results of monitoring and sampling (1) the dry and wet weather sanitary stations, (2) the combined sewer overflow stations, and (3) the storm water discharge stations. The characteristics of each monitoring point is described briefly herein, and a detailed description of each point is contained in Appendix A.

The results of the monitoring and sampling program are tabulated in Appendix B. This appendix also includes the pounds of the various constituents for those samples where corresponding flow data was available. An interpretation of the results is provided herein, based on observations in the field as well as laboratory analyses.

TYPES OF SAMPLES

The types of flows which were monitored fall into one of the following five general categories:

		Station Designation
1.	Dry Weather Sanitary (combined) Flow	D-
2.	Wet Weather Sanitary (combined) Flow	W-
3.	Combined Sewer Overflow	O-
4.	Separate Storm Water Discharges	S-
5.	Rivers and Creeks	R-

The primary purpose of this study was to ascertain and evaluate the magnitude of the river pollution problem attributable to combined sewer overflow, hence items "3" and "5" above. Dry and wet weather sanitary sampling, though questioned at first as being unnecessary, were left in the program and ultimately proved to be of considerable value. This data was used to determine domestic and industrial load impacts for overflow relief sewer design and was especially valuable in assessing the magnitude of the unusually high infiltration flows measured in the Des Moines sewer system. This data, together with the separate storm water discharge data, provided comparative values for evaluating combined sewer overflow pollutant concentrations.

DRY AND WET WEATHER SANITARY FLOWS

Dry weather flows were sampled at eleven locations as listed below. A summary description of each point is contained in Table 4. Also, the points may be located by reference to Figure 13.

The stations sampled and the respective interceptors or drainages served were as follows:

Station	Interceptor and Location
D-1*(W-1)	West Side Interceptor @ Scott Street
D-1A*	Ingersoll Run Sewer @ 22nd & High
D-1B*	Closes Creek Trunk @ Harding & Prospect
D-2	Southwest Outfall Sewer @ Raccoon River Siphon
D-3*(W-3)	East Side Interceptor @ E. 1st & Racoon
D-4*	East 18th St. Interceptor @ Maury St.
D-5*(W-5)	South Side Trunk @ Des Moines River Siphon
D-6*	Main Outfall @ Wastewater Treatment Plant
D-7	Storm Outlet @ Cornell & Aurora
D -8	Fraley Ditch @ E. 30th & Court (storm sewer outlet)
D-9	Four Mile Trunk @ 33rd & E. Granger

*Combined Sewer

TABLE 4

SUMMARY DESCRIPTION OF MONITORING POINTS

STATION	SEWER SYSTEM	STATION LOCATION	DRAINAGE AREA ABOVE STATION					FLOW MONITORING		SEWER SIZE	TYPE REGULATOR
			EST. 1969 POPULATION	ACRES	POPULATION DENSITY	EST. % INDUSTRIAL	EST. % COMBINED SEWERS	METHOD OF FLOW MEASUREMENT	MONITORING EQUIPMENT		
D-1	WEST SIDE INTERCEPTOR SEWER	SCOTT STREET SIPHON OUTLET	79,000	8,681	9.1	3.0	18	2 WEIRS	FLOAT RECORDER	60"	N
W-1	SAME AS D-1	SAME AS D-1	79,000	8,681	9.1	3.0	18	NONE	BUBBLER	60"	N
W-1A	SAME AS D-1	SCOTT STREET SIPHON INLET	79,000	8,681	9.1	3.0	18	NONE	BUBBLER	60"	O.P
W-1B (1)	SAME AS D-1	WEST 1ST & ELM	79,000	8,681	9.1	3.0	18	CURRENT METER	NONE	60"	N
D-1A	INGERSOLL RUN SEWER	22ND & HIGH	10,100	927	10.9	0.1	81	WEIR	FLOAT RECORDER	5'X10'	O.W
D-1B	CLOSES CREEK TRUNK	ABAND. PUMPSTATION NO PROSPECT	12,300	1,673	7.3	0	6	WEIR	FLOAT RECORDER	30"	N
D-2	SOUTHWEST OUTFALL SEWER	RACOON RIVER SIPHON OUTLET	54,690	15,720	3.5	1.0	0	WEIR	FLOAT RECORDER	78"	N
W-2	SAME AS D-2	SAME AS D-2	54,690	15,720	3.5	1.0	0	WEIR (2)	BUBBLER	78"	N
D-3	EAST SIDE INTERCEPTOR SEWER	EAST 1ST & RACOON	16,400	2,240	7.3	2.4	24	WEIR	FLOAT RECORDER	48"	N
W-3	SAME AS D-3	SAME AS D-3	16,400	2,240	7.3	2.4	24	WEIR (2)	BUBBLER	48"	N
D-4	EAST 18TH STREET INTERCEPTOR	EAST 18TH & MAURY	17,300	3,024	5.7	6.5	3	WEIR	FLOAT RECORDER	66"	N
D-5	SOUTH SIDE TRUNK SEWER	DES MOINES RIVER SIPHON INLET	15,300	2,061	7.4	1.1	7	WEIR	FLOAT RECORDER	36"	O.W
W-5	SAME AS D-5	SAME AS D-5	15,300	2,061	7.4	1.1	7	NONE	BUBBLER	36"	O.W
W-5A	SAME AS D-5	DES MOINES RIVER SIPHON OUTLET	15,300	2,061	7.4	1.1	7	NONE	BUBBLER	24"X45"	N
D-6	DES MOINES WWTP	DES MOINES WWTP	239,720	46,167	5.2	2.2	7	WWTP FLOW METER	(3)	78"X84"	O.W
D-7	DRAINAGE DITCH	CORNELL & AURORA	UNKNOWN - OMITTED FROM PROGRAM AFTER INITIAL SAMPLING				0	WEIR	BUBBLER	DITCH	N.A
D-8	FRALEY DITCH	EAST 31ST & COURT					0	WEIR	FLOAT RECORDER	DITCH	N.A
D-9	FOURMILE TRUNK SEWER	E. GRANGER ABOVE WWTP	35,600	6,558	5.4	1.4	0	WEIR	BUBBLER	48"	N
W-9	SAME AS D-9	SAME AS D-9	35,600	6,558	5.4	1.4	0	WEIR	BUBBLER	48"	N
O-2	CLOSES CREEK DRAINAGE	BETWEEN HARDING & PROSPECT	12,300	1,673	7.3	0	6	STAGE DISCHARGE	BUBBLER	DITCH	N.A
O-3	CLOSES CREEK & N.W. OUTFALL	PROSPECT & HICKMAN	30,300	4,053	7.5	0	6	NONE	BUBBLER	36"	O.W
O-4	WEST SIDE INTERCEPTOR SEWER	2ND & FRANKLIN	38,000	4,800	7.9	0	10	NONE	STICK GAGE	60"	O.W
O-5	WEST SIDE INTERCEPTOR SEWER	2ND & GRAND	46,500	5,600	8.3	0	25	NONE	BUBBLER	60"	O.P
O-6	SAME AS O-5	451 FEET BELOW O-5	46,500	5,600	8.3	0	25	HYDRAULIC GRADIENT	BUBBLER	60"	O.W
O-7	WEST SIDE INTERCEPTOR SYSTEM	W.S. STORM BOX @ SCOTT ST	79,000	8,681	9.1	1	33	CURRENT METER AND	BUBBLER	60"	O.P
O-7A	SAME AS O-7	W.S. STORM BOX @ ELM ST	71,800	7,900	9.1	1	33	HYDRAULIC GRADIENT	BUBBLER	60"	O.P
O-7B	SAME AS O-7	W.S. STORM BOX @ GRAND AVE	59,200	6,500	9.1	1	33		BUBBLER	60"	O.P
O-8	INGERSOLL RUN SEWER	OVERFLOW OUTLET @ 17TH & RAILRD.	14,700	1,350	10.9	0.1	69	STAGE DISCHARGE	BUBBLER	5'X13'	N.A
O-8A	SAME AS O-8 & D-1A	SAME AS D-1A	10,100	927	10.9	0.1	81	NONE	BUBBLER	5'X13'	O.W
O-11	20TH ST. STORM SEWER	20TH ST. & DEAN LAKE	12,500	1,170	10.7	5	0	WEIR	BUBBLER	4'X5'	N.A
O-14	MAIN OUTFALL @ WWTP	WWTP BYPASS	239,720	46,167	5.2	2.2	7	WEIR	BUBBLER	78"X84"	O.W
S-1	THOMPSON AVE. STORM SEWER	BIRDLAND PARK LAGOON	2,300	310	7.4	0	0	WEIR	BUBBLER	5'X4'	N.A.
S-2	UNIVERSITY BOX-TUNNEL	W 1ST & UNIVERSITY	1,800	193	9.3	0	0	WEIR	FLOAT RECORDER	5'X6'	N.A
S-3	CUMMINS PARKWAY STORM SEWER	63RD & CUMMINS PARKWAY	1,900	356	5.3	0	0	WEIR	BUBBLER	4'X6'	N.A.

NOTES:

(1) STATION WAS USED ONLY FOR FLOW MEASUREMENT

(2) DURING HIGH FLOW WEIR WAS AFFECTED BY DOWNSTREAM SURCHARGE

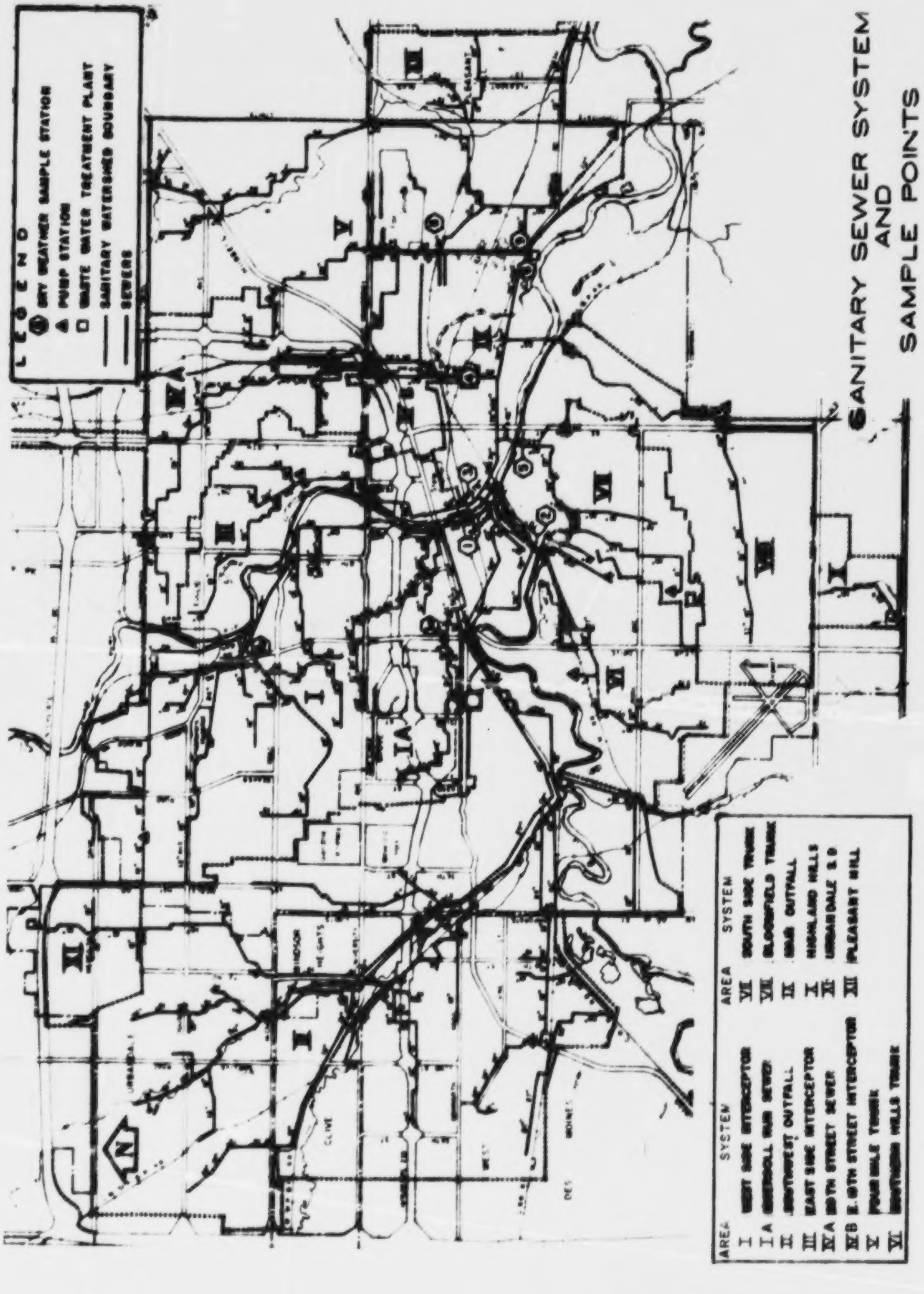
(3) COMPOSITES COLLECTED AND ANALYZED BY WWTP PERSONNEL

STATION CODE:

D - DRY WEATHER
W - WET WEATHER
O - OVERFLOW
S - STORM SEWER

TYPE REGULATOR CODE:

N - NONE
O.W.- OVERFLOW WEIR
O.P.- OVERFLOW PIPE
N.A.- NOT APPLICABLE


LEGEND
DRY WEATHER SAMPLE STATION
PUMP STATION
WASTE WATER TREATMENT PLANT
SANITARY WATERSHED BOUNDARY
SEWERS
N
CLIVE
WEST DES MOINES
URBANDALE
WINDSOR HEIGHTS
AREA SYSTEM
I WEST SIDE INTERCEPTOR
I A INGERSOLL RUN SEWER
II SOUTHWEST OUTFALL
III EAST SIDE INTERCEPTOR
IV A 30TH STREET SEWER
IV B E. 18TH STREET INTERCEPTOR
V FOUR MILE TRUNK
VI [illegible] HILLS TRUNK
AREA SYSTEM
VII SOUTH SIDE TRUNK
VIII BLOOMFIELD TRUNK
IX [illegible] OUTFALL
X HIGHLAND HILLS
XI URBANDALE S.D.
XII PLEASANT HILL

SANITARY SEWER SYSTEM AND SAMPLE POINTS

FIGURE 13

Dry weather flow measurement and sampling was accomplished for all stations except D-9 during October and November, 1968, but analytical data is not presented. For reasons unknown, the analytical data from these initial samplings was extremely erratic and considered unreliable. Therefore all stations except D-4, D-7 and D-8 were resampled. Station D-7 was determined to be a separate storm drain and was not resampled because the initial sampling indicated an absence of sanitary wastes. Station D-4 had considerable quantities of industrial wastes but was not resampled because in January, 1969, the City began an industrial waste control program which, when implemented, would materially alter the loadings in these sewers. In lieu of resampling, data from the City's industrial waste sampling program was used to evaluated present and future industrial waste loads. Station D-8, a storm drain, carried a small amount of industrial wastes. This was removed as a result of the City's industrial waste control program.

Station D-6 is the wastewater treatment plant. Sampling at this station was done by WWTP personnel as part of their routine sampling program. Unfortunately, the plant fow meter was out of calibration during the October 1968-Februray 1969 period of dry weather sanitary sampling, so comparative total plant loads could not be computed. For this reason, plant fow and analytical data for August, September and October, 1969, were obtained and evaluated.

Table 5 is a summary of the sanitary and industrial loads developed from a combination of the dry weather sampling and the industrial waste data obtained from the City.

Because of physical and hydraulic limitations at the wet weather sample stations, flow measurement could not be obtained. Weirs used for dry weather sampling were submerged. Attempts to correlate flow with head loss across river siphons proved extremely erratic, possibly because of constantly changing sediment restrictions in the siphons. Also, surcharge from the main outfall caused a constantly changing hydraulic gradient at all the wet weather stations, sometimes showing a negative gradient which would indicate reverse flow.

Since accurate flow data could not be obtained, wet weather samples were analyzed either as grab samples or as composite samples based on estimates of flow made from evaluation of relative surcharge elevations. The analytical data obtained is considered to be useful as a guide for evaluating the effect of diluting sanitary flows with storm runoff. A comparison of dry and wet weather data is shown in Table 6. BOD and nutrient concentrations were generally greater in the dry weather flows, while wet weather flows contained greater concentrations of suspended solids. This is due to the influence of storm runoff which typically contains higher suspended solids and lower BOD's than dry weather sanitary flows. Wet weather sanitary data was also obtained as part of the overflow monitoring described in subsequent paragraphs and in Table 4. Useful data was obtained from:

Station 0-3	West Side Interceptor @ Prospect & Hickman
Station 0-6	West Side Interceptor @ Grand Avenue
Station 0-8A	Ingersoll Run Sewer @ 22nd & High

TABLE 5

SUMMARY OF DRY WEATHER SANITARY AND INDUSTRIAL WASTE LOADS

DRY WEATHER SANITARY SAMPLING	
For Stations D-1, D-2, D-3, D-5 and D-9 Combined	
Population in Sample Area	200,900
Average Daily Flow	18.40 MGD
Per Capita Flow	91.50 GPD
Average Daily BOD	33,229 LBS
Per Capita BOD	0.165 LBS
Average Daily TSS	32,747 LBS
Per Capita TSS	0.163 LBS
WASTEWATER TREATMENT PLANT DATA	
August-September-October 1969 (13 Composite Periods)	
Average Daily Flow	35.3 MGD
Average Daily BOD	95,800 LBS
Average Daily TSS	106,000 LBS
FOR 247,920 PRESENT POPULATION	
Average Daily Domestic Flow, @ 95 GPCD	23.6 MGD
Apparent Average Daily Industrial Flow	11.7 MGD
Average Daily Domestic BOD @ 0.165 lbs/cap.	40,900 LBS
Apparent Industrial Contribution	54,900 LBS
Average Daily Domestic TSS @ 0.163 lbs/cap.	41,400 LBS
Apparent Industrial Contribution	64,600 LBS

TABLE 6

DRY WEATHER VS. WET WEATHER FLOWS IN COMBINED SEWERS

	D-1		W-1		D-3		W-3		D-5		W-5	
Test	No. Tests	Avg. MG/L	No. Tests	Avg. MG/L	No. Tests	Avg. MG/L	No. Tests	Avg. MG/L	No. Tests	Avg. MG/L	No. Tests	Avg. MG/L
B.O.D.	9	212	8	63	10	169	4	128	5	216	4	138
T.S.S.	4	228	8	384	6	153	4	386	3	390	3	1079
V.S.S.	4	190	8	143	6	117	4	175	3	250	3	366
NH_3.N	6	21.30	5	7.20	5	23.81	3	22.67	3	31.13	2	8.48
NO_2.N	7	0.11	5	0.12	5	0.12	3	0.17	3	0.20	2	0.01
NO_3.N	6	0.98	5	1.16	3	0.13	4	0.44	1	0.16	2	0.06
T.PO_4	5	20.13	5	12.79	5	14.84	2	14.68	3	17.70	2	16.20
O.PO_4	5	11.49	5	8.06	5	9.08	4	6.59	3	13.21	2	7.08

Note: The data for W-5 is for only one runoff period.

The data for W-1 and W-3 is for 4 and 3 runoffs respectively.

A summary of the overflow monitoring results is given in Table 7. As a matter of interest, comparative average BOD for wet weather sanitary flows are tabulated below for the three stations.

Station 0-3	2 runoff periods,	Avg. BOD = 69 mg/l
Station 0-6	3 runoff periods,	Avg. BOD = 72 mg/l
Station 0-8A	4 runoff periods,	Avg. BOD = 69 mg/l

COMBINED SEWER OVERFLOWS AND STORM WATER DISCHARGES

Combined sewer overflows were monitored at five locations and storm water discharges at three locations.

Stations sampled and sewers or drainage areas served were as follows:

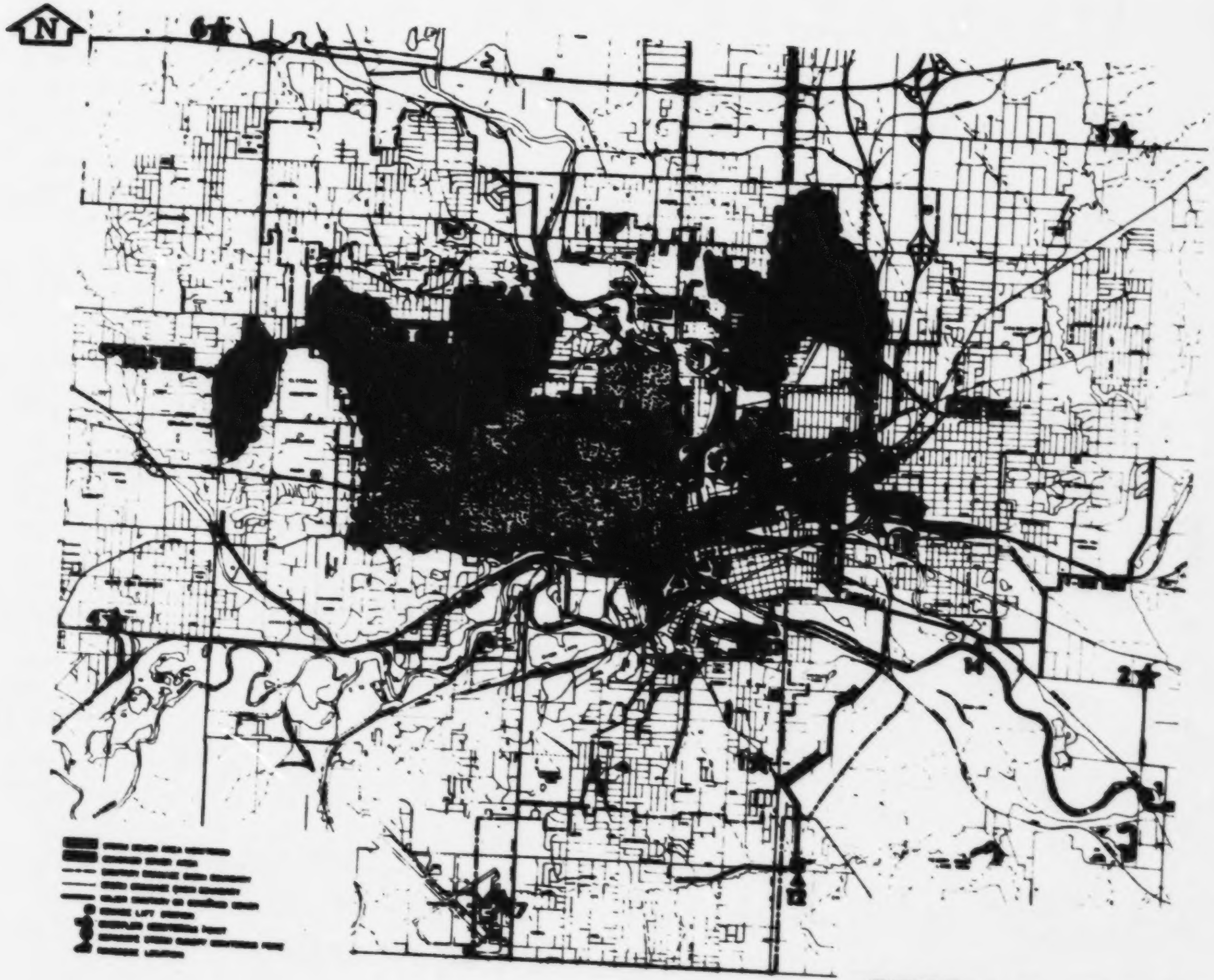
Station	Sewer or Area
0-2	Closes Creek overflow and drainage area
0-3	West Side Interceptor @ Prospect & Hickman
0-6	West Side Interceptor @ Grand Avenue
0-8	Ingersoll Run overflow @ outlet
0-8A	Ingersoll Run sewer @ 22nd & High
S-1	Thompson Avenue Storm Sewer
S-3	Cummins Parkway Storm Sewer
0-11	20th Street Storm Sewer (separate sewer)

A summary description of each point is contained in Table 4. Figure 14 shows the location of these points, and a detailed description of each point is located in Appendix A.

Figure 15 shows typical overflow structures located at Stations 0-5 and 0-6. Both are located on the West Side Interceptor immediately above Grand Avenue.

TABLE 7
COMBINED SEWAGE OVERFLOW & STORM WATER DISCHARGES

Station	B.O.D.	Suspended Solids		Nitrogens			Phosphates	
		T.S.S.	V.S.S.	NH_3.N	NO_2.N	NO_3.N	T.PO_4	O.PO_4
0-2 (12 Storms)								
No. Tests	19	19	19	17	17	17	14	17
Avg. MG/L	44	495	95	1.21	0.03	0.88	2.20	1.25
0-3 (2 Storms)								
No. Tests	3	3	3	1	1	1	1	1
Avg. MG/L	69	144	77	4.53	0.08	0.26	11.72	8.25
0-6 (5 Storms)								
No. Tests	10	9	9	7	7	6	4	5
Avg. MG/L	95	592	181	9.42	0.14	0.63	9.88	9.24
0-7 (5 Storms)								
No. Tests	9	8	8	8	8	8	7	7
Avg. MG/L	50	195	62	1.84	0.01	1.15	7.10	3.02
0-8 (12 Storms)								
No. Tests	21	20	20	18	18	18	13	12
Avg. MG/L	68	410	142	3.22	0.03	1.07	6.96	5.08
0-8A (4 Storms)								
No. Tests	7	5	5	5	5	5	0	5
Avg. MG/L	77	303	101	4.94	0.03	0.57	-	6.05
S-1 (17 Storms)								
No. Tests	25	32	30	9	9	9	7	9
Avg. MG/L	48	315	99	1.99	0.15	1.11	1.25	0.18
S-3 (20 Storms)								
No. Tests	24	24	21	11	11	11	7	10
Avg. MG/L	63	578	106	1.60	0.03	1.47	0.93	0.43
0-11 (20 Storms)								
No. Tests	35	33	32	29	29	29	21	26
Avg. MG/L	56	404	110	2.30	0.04	0.96	2.23	0.57

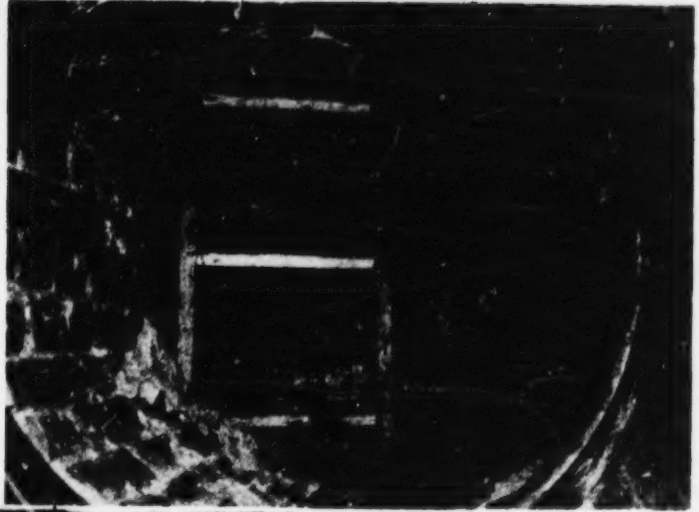


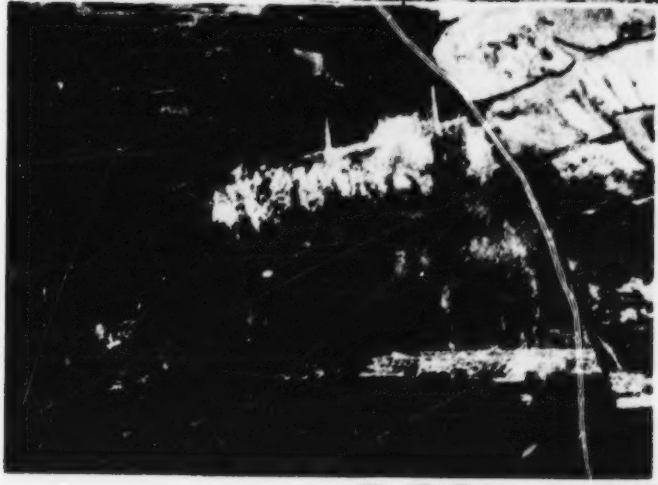
STORM & COMBINED SEWER SAMPLING POINTS

FIGURE 14

LOOKING DOWN AT 36" OVERFLOW FROM 60" WEST SIDE INTERCEPTOR AT STATION O-5

LOOKING INTO 60" WEST SIDE INTERCEPTOR FROM 36" OVERFLOW AT STATION O-5





BRICK AND MORTAR OVERFLOW AT STATION O-6

TYPICAL OVERFLOW STRUCTURES

FIGURE 15

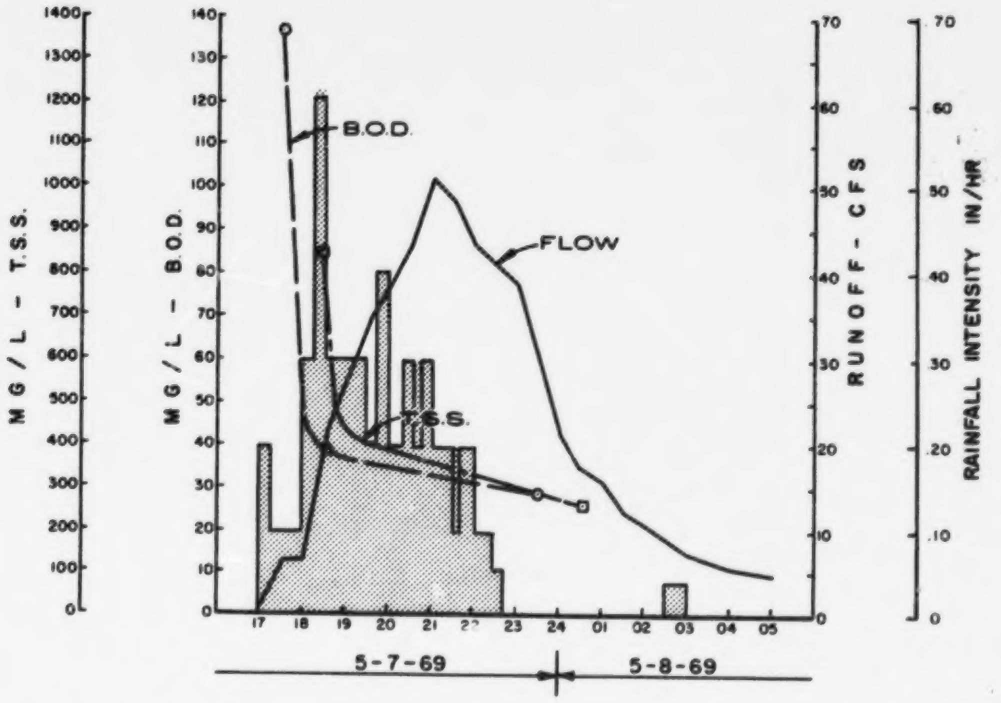
Station 0-5 consists of a brick overflow sewer leading directly from the interceptor; at 0-6 the overflow passes over a brick weir constructed in the side of the interceptor sewer. One of the original goals of this study was to determine the actual quantity of combined sewer overflows and pollutant material generated from runoffs of various magnitudes. In retrospect, this goal was idealistic. In many instances in the system studied, the actual volume of overflow often could not be determined. Excessive infiltration, almost constant surcharge during the spring and summer, and limited hydraulic capacity often interfered with attempts to monitor overflow. When runoff water reaches a sewer inlet, it may enter there only to emerge at some downstream inlet to become overland flow. Generally, this condition was predicted for the design storms and other high intensity storms, but was not considered to be of great significance for most of the lower intensity storms monitored during the study. The surcharge conditions and the long period of high river stage did, however, interfere to a great extent with monitoring of major combined sewer overflows.

For these reasons, theoretical computations of overflow quantities were considered more reliable in some cases than the hydraulic measurement obtained. Measured pollutant concentrations were used in determining predicted overflow quality. Predictions of area-wide overflow quantities were based on standardized rainfall curves developed for the study area and the hydraulic capacities of the present sewer system. The results of these calculations are discussed in Section VIII and X.

Stations 0-2, 0-8, S-1, S-3 and 0-11 each represent the only point of overflow or storm water discharge from their respective watersheds. Also, the boundaries of their contributing sewer area could be determined within an allowable margin of error. For these stations, unit values of runoff and pollutant quantities were determined. These data are presented graphically in Figures 16 through 22. The data lends support to the "first flush" theory. In almost all cases, BOD and TSS concentrations decreased with time with little or no relation to the flow pattern. Also, volumetrically, the BOD and TSS runoff almost always "ran ahead" of flow. Similarly, chloride concentrations during snowmelt runoff indicate a high initial flush. Serial data on nutrient concentrations was not adequate to determine if the "first flush" theory relates to these pollutants also, however, logic would suggest that it does.

Figure 23 shows in general the relationship between the volume of rainfall and the volume of runoff. In Figure 24, unit pollutant values are plotted for the overflow stations (0-2 and 0-8) and the storm sewer stations (S-1, S-3 and 0-11). Reference to Table 7 shows the average constituent concentrations for each of the overflow and storm water stations. Composite and intermittent grab samples were taken throughout runoff periods in order to more closely obtain the true characteristics of the

RUNOFF CHARACTERISTICS



PERTINENT DATA

RUNOFF PERIOD: 1700 HRS 5-7-69
TO 1600 HRS 5-8-69

TOTAL RUNOFF: 23.2 AC. FT.
0.238 INCHES

RAINFALL PERIOD: 1650 HRS 5-7-69
(AT R.G. No. 5) TO 2240 HRS 5-7-69

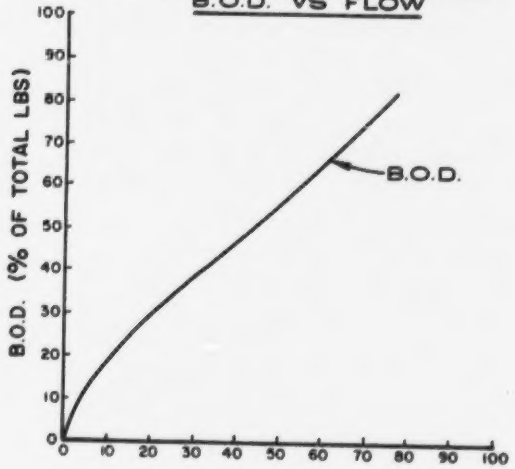
TOTAL RAINFALL = 1.14"

VOLUMETRIC COEFFICIENT OF
RUNOFF = 0.208

ANALYZED RUNOFF: 22.9 AC. FT.
(99% OF TOTAL)

COMPOSITE PERIODS (2)
(1) 1730 HRS 5-7-69 TO
0030 HRS 5-8-69
COMPOSITE B.O.D. = 31 MG/L
T.S.S. = 381 MG/L
(2) 0130 HRS TO 0830 HRS 5-8-69
COMPOSITE B.O.D. = 22 MG/L
T.S.S. = 97 MG/L

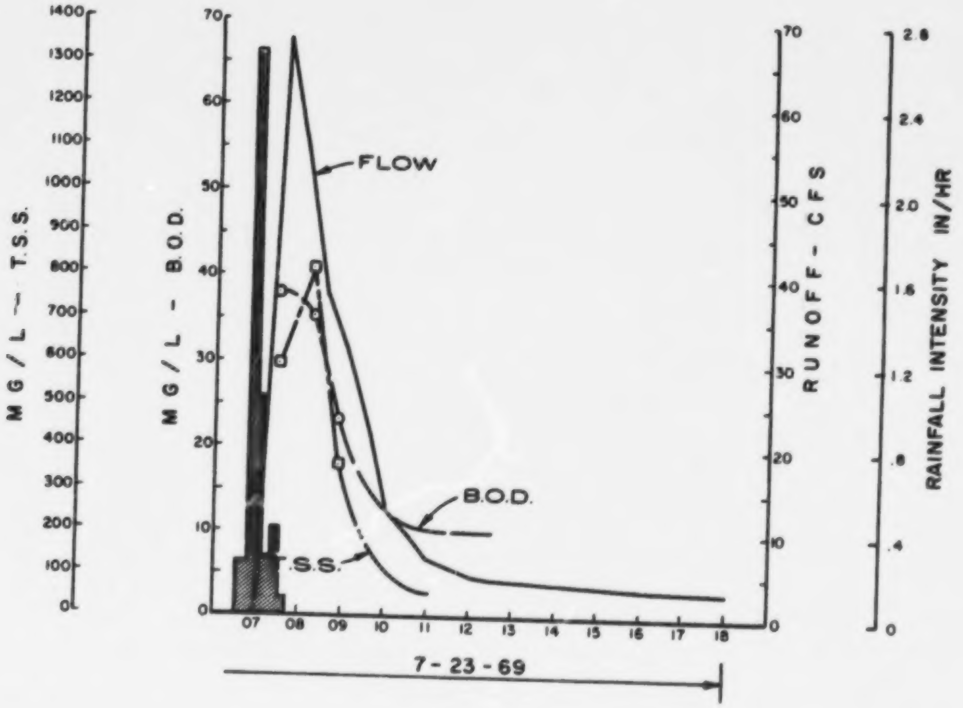
VOLUMETRIC RELATIONSHIP
B.O.D. VS FLOW



RUNOFF CHARACTERISTICS
STATION O-11
20TH STREET STORM SEWER

FIGURE 16

RUNOFF CHARACTERISTICS



PERTINENT DATA

RUNOFF PERIOD: 0700 HRS
TO 1800 HRS 7-23-69

TOTAL RUNOFF: 9.30 AC. FT.
0.097 INCHES

RAINFALL PERIOD: 0630 HRS
(AT R.G. No.5) TO 0745 HRS

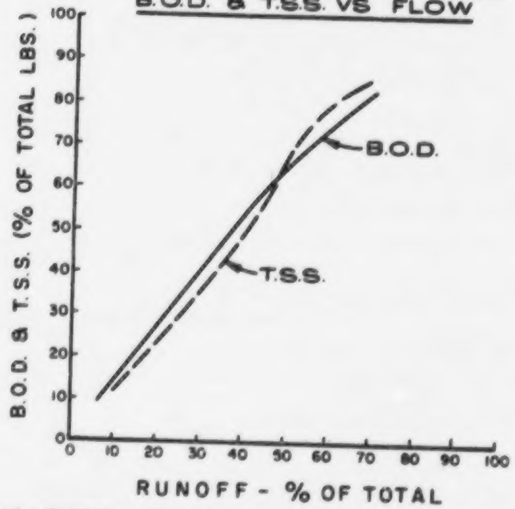
TOTAL RAINFALL = 0.69"

VOLUMETRIC COEFFICIENT OF
RUNOFF = 0.141

ANALYZED RUNOFF: 5.90 AC. FT.
(64% OF TOTAL)

RUNOFF SAMPLES: 3 GRABS
(1) 0725 HRS: B.O.D. = 37.5 MG/L
T.S.S. = 588 MG/L
(2) 0815 HRS: B.O.D. = 35.0 MG/L
T.S.S. = 808 MG/L
(3) 0850 HRS: B.O.D. = 23.1 MG/L
T.S.S. = 356 MG/L

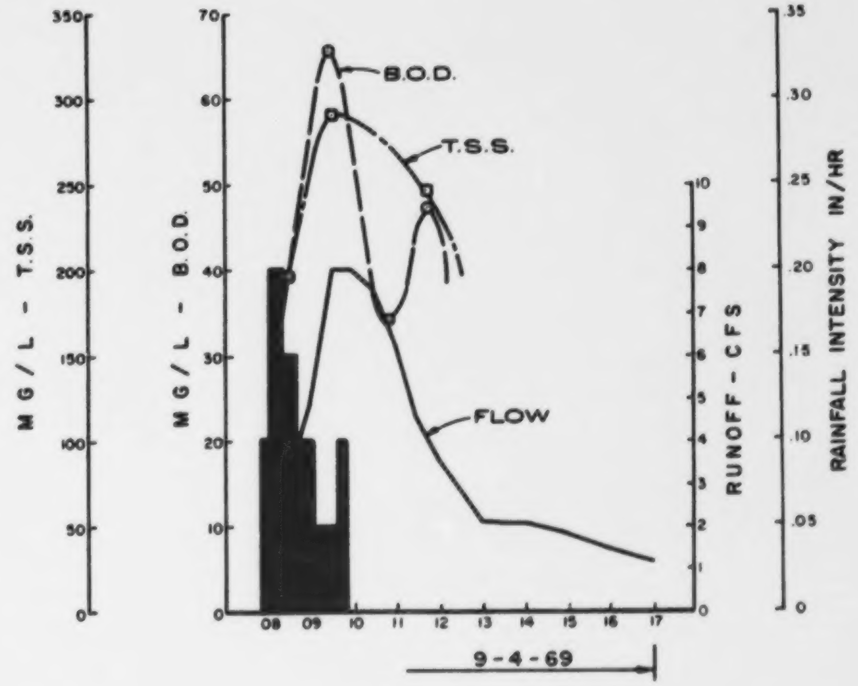
VOLUMETRIC RELATIONSHIP
B.O.D. & T.S.S. VS FLOW



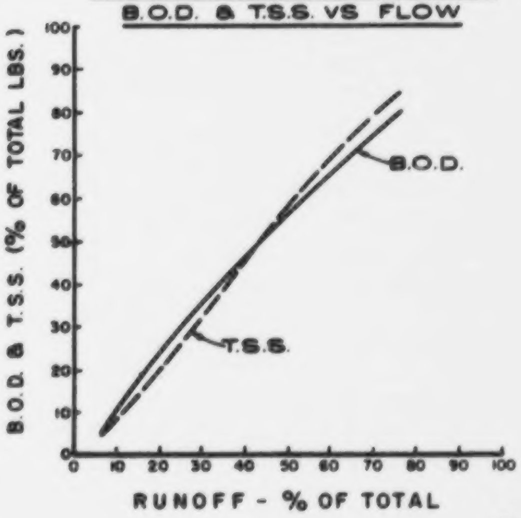
RUNOFF CHARACTERISTICS
STATION O-11
20TH STREET STORM SEWER

FIGURE 17

RUNOFF CHARACTERISTICS



VOLUMETRIC RELATIONSHIP
B.O.D. & T.S.S. VS FLOW



PERTINENT DATA

RUNOFF PERIOD: 0800 HRS
TO 1700 HRS 9-4-69

TOTAL RUNOFF: 1.85 AC. FT.
0.018 INCHES

RAINFALL PERIOD: 0745 HRS TO
(AT R.G. No.5) 0945 HRS

TOTAL RAINFALL = 0.22"

VOLUMETRIC COEFFICIENT OF
RUNOFF = 0.055

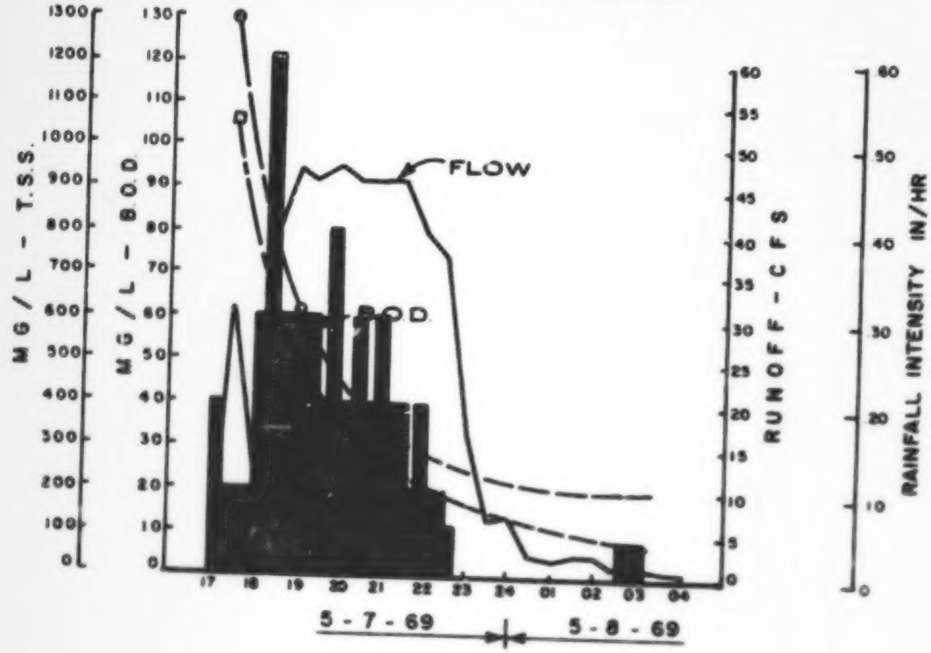
ANALYZED RUNOFF: 1.47 AC. FT.
(80% OF TOTAL)

RUNOFF SAMPLES: 4 GRABS
(1) 0830 HRS: B.O.D. = 40.0 MG/L
T.S.S. = 196 MG/L
(2) 0930 HRS: B.O.D. = 65.0 MG/L
T.S.S. = 280 MG/L
(3) 1045 HRS: B.O.D. = 34.0 MG/L
(4) 1145 HRS: B.O.D. = 47.0 MG/L
TSS = 247 MG/L

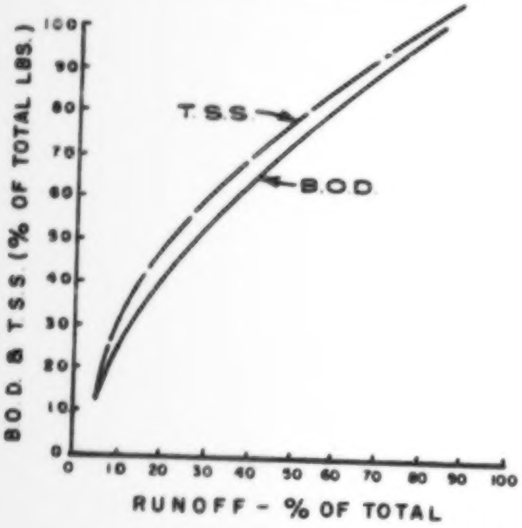
RUNOFF CHARACTERISTICS
STATION O-11
20TH STREET STORM SEWER

FIGURE 18

RUNOFF CHARACTERISTICS



VOLUMETRIC RELATIONSHIP
B.O.D. & T.S.S. VS FLOW



PERTINENT DATA

RUNOFF PERIOD 1700 HRS. 5-7-69
TO 0400 HRS 5-8-69

TOTAL RUNOFF: 19.29 AC. FT
0.171 INCHES

RAINFALL PERIOD: 1650 HRS 5-7-69
(AT R.G. No.5) TO 2240 HRS 5-7-69

TOTAL RAINFALL = 1.14"

VOLUMETRIC COEFFICIENT OF
RUNOFF - 0.150

ANALYZED RUNOFF: 18.75 AC. FT
(97.4 OF TOTAL)

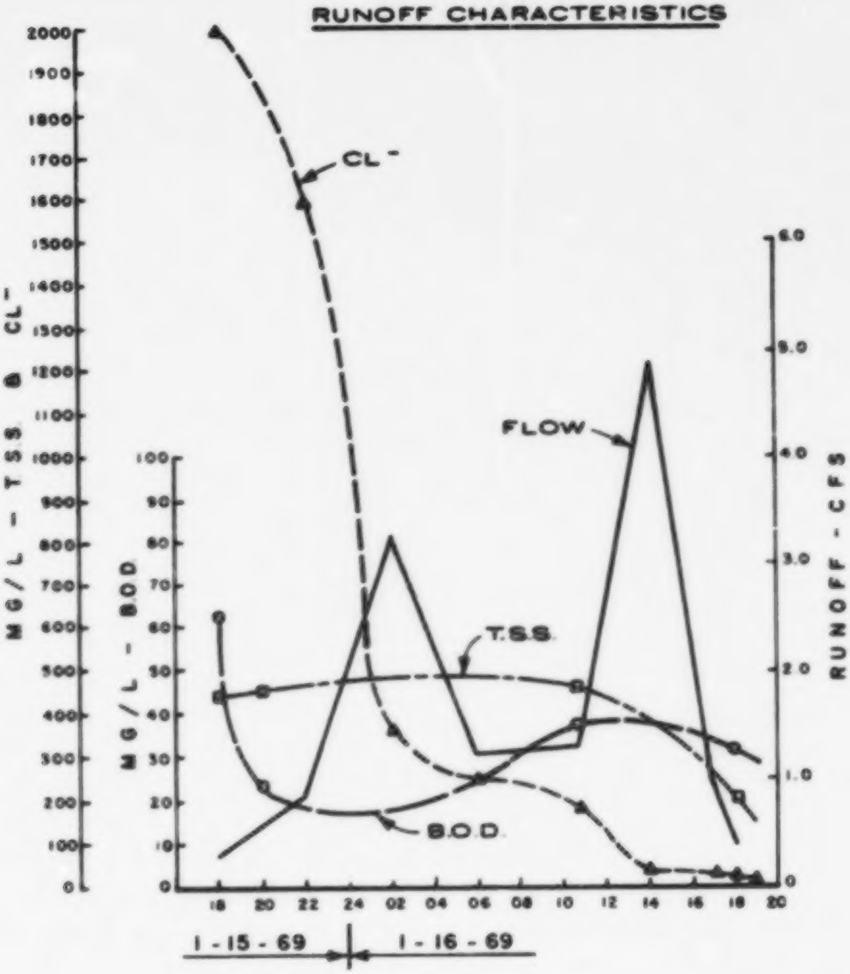
COMPOSITE PERIOD: 1700 HRS -
2300 HRS 5-7-69

COMPOSITE B.O.D. = 44.8 MG/L
T.S.S. = 343 MG/L

RUNOFF CHARACTERISTICS
STATION O-8
INGERSOLL RUN OVERFLOW AT OUTLET

FIGURE 19

RUNOFF CHARACTERISTICS



PERTINENT DATA

RUNOFF PERIOD : 1800 HRS 1-15-69
TO 1800 HRS 1-16-69

PRECIPITATION : NONE - SNOWMELT

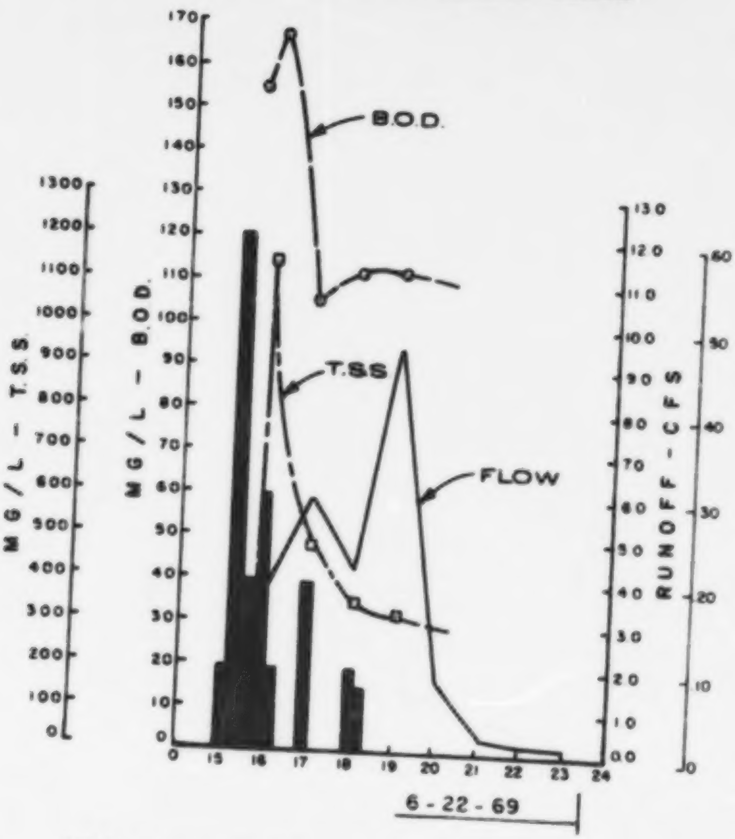
COMPOSITE PERIOD : SAME AS ABOVE

COMPOSITE B.O.D. = 31 MG/L
T.S.S. = 302 MG/L
CL⁻ = 100 MG/L

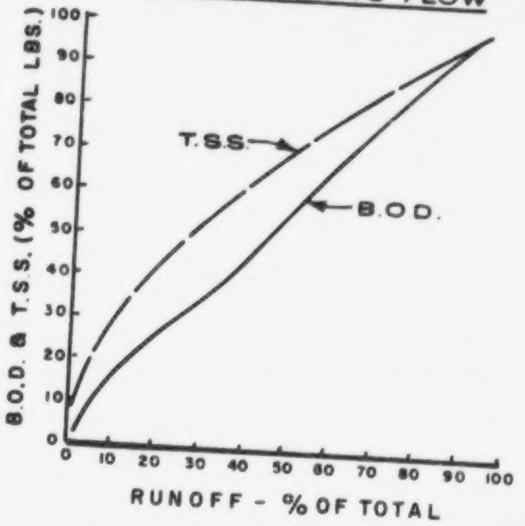
RUNOFF CHARACTERISTICS
STATION S-3
CUMMINS PARKWAY STORM SEWER

FIGURE 20

RUNOFF CHARACTERISTICS



VOLUMETRIC RELATIONSHIP
B.O.D. & T.S.S. VS FLOW



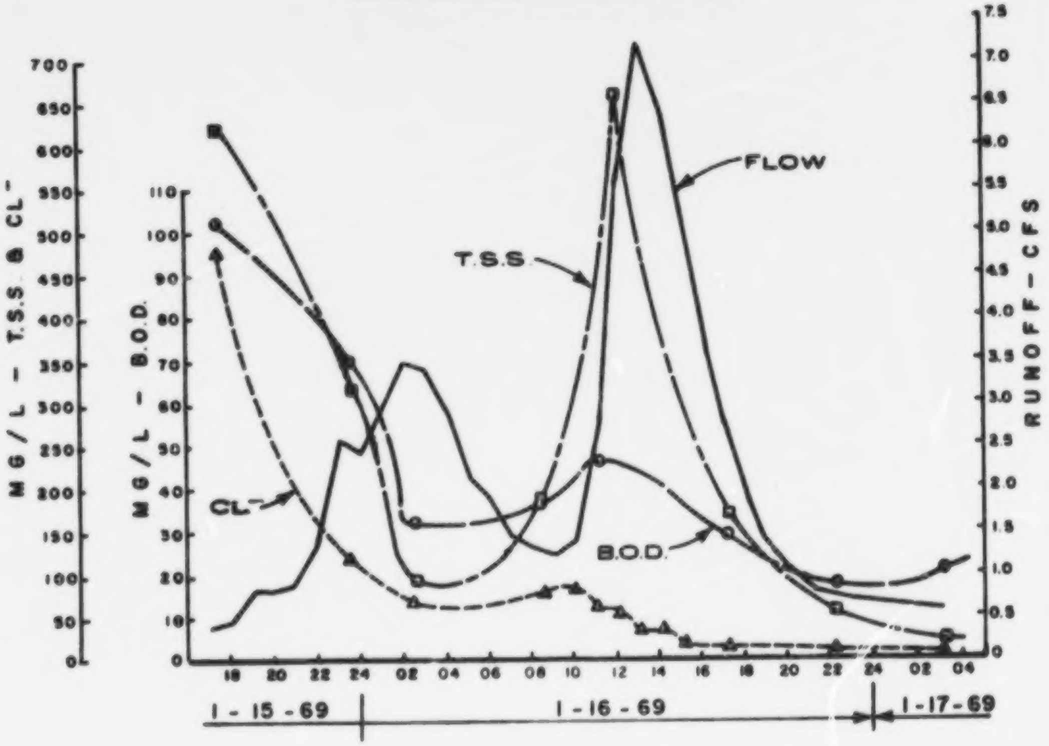
PERTINENT DATA

RUNOFF PERIOD: 1540 HRS. – 2300 HRS.
6 - 22 - 69
TOTAL RUNOFF: 2.08 AC. FT.
0.07 INCHES
RAINFALL PERIOD: 1500 HRS. – 1800 HRS.
TOTAL RAINFALL: 0.42"
(AT R.G. No.5)
VOLUMETRIC COEFFICIENT OF
RUNOFF = 0.167
ANALYZED RUNOFF: 1.92 AC. FT.
(92 % OF TOTAL)
RUNOFF SAMPLES: 5 GRABS
(1) 1540 HRS.: B.O.D. = 154 mg/l
T.S.S. = 99 mg/l
(2) 1600 HRS.: B.O.D. = 166 mg/l
T.S.S. = 1146 mg/l
(3) 1700 HRS.: B.O.D. = 105 mg/l
T.S.S. = 476 mg/l
(4) 1800 HRS.: B.O.D. = 111 mg/l
T.S.S. = 340 mg/l
(5) 1900 HRS.: B.O.D. = 111 mg/l

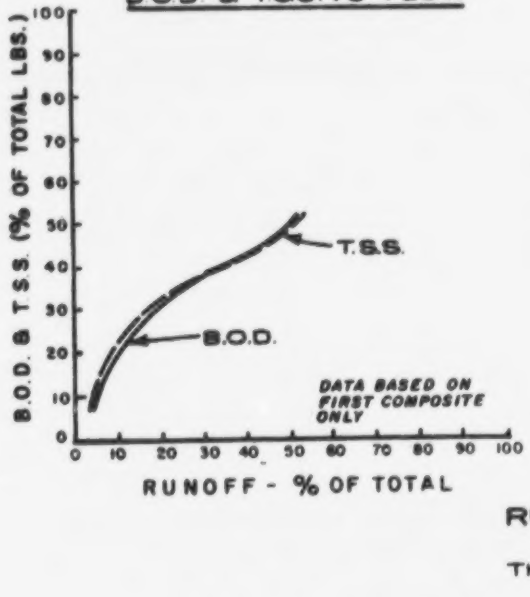
RUNOFF CHARACTERISTICS
STATION S – 3
CUMMINS PARKWAY STORM SEWER

FIGURE 21

RUNOFF CHARACTERISTICS



VOLUMETRIC RELATIONSHIP
B.O.D. & T.S.S. VS FLOW



PERTINENT DATA

RUNOFF PERIOD: 1800 HRS 1-15-69
TO 0300 HRS 1-17-69

TOTAL RUNOFF: 6.09 AC. FT
0.246 INCHES

PRECIPITATION: NONE - SNOWMELT

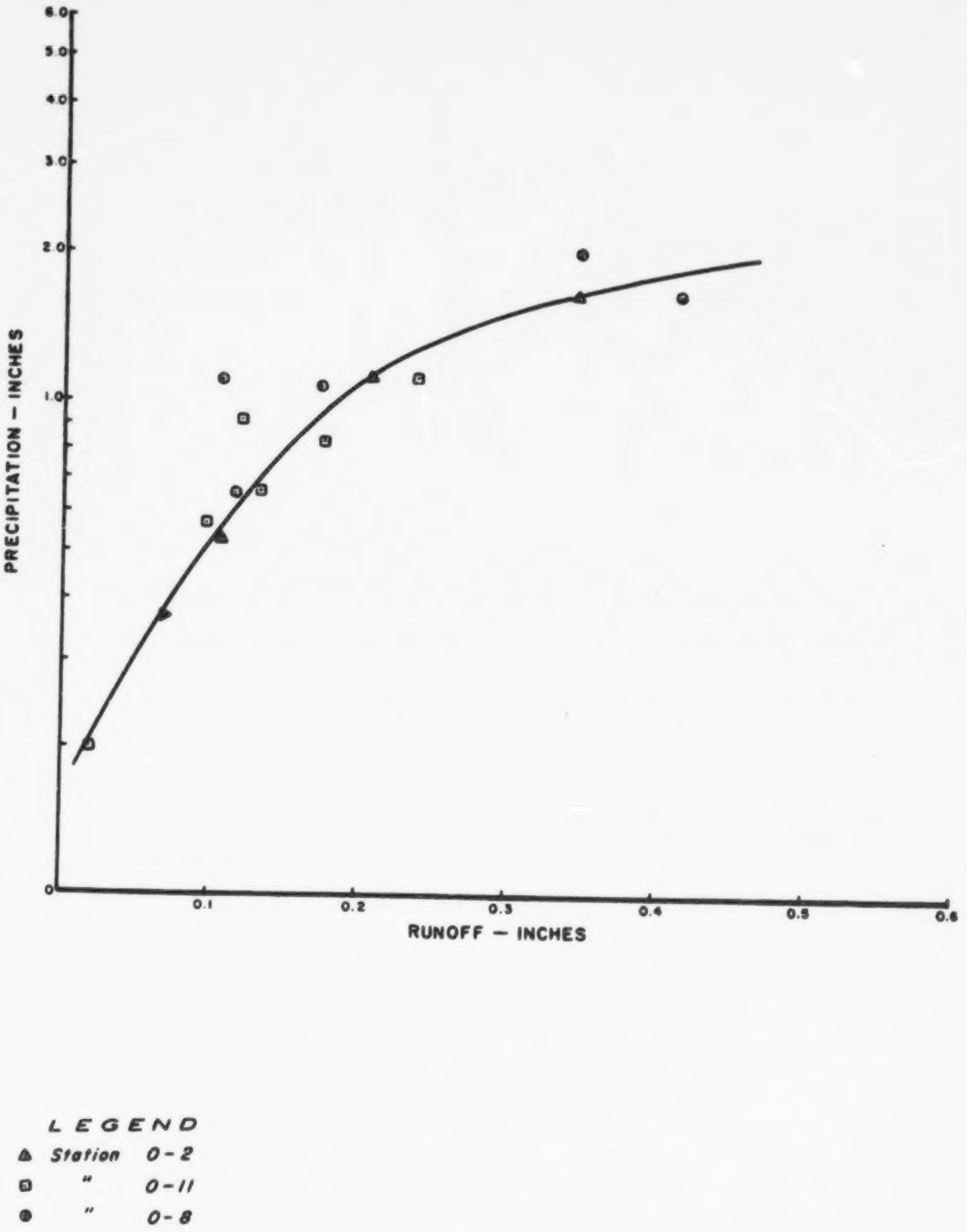
COMPOSITE PERIODS (2):

(1) 1700 HRS 1-15 TO 0900 HRS 1-16
COMPOSITE B.O.D. = 46 MG/L
T.S.S. = 223 MG/L
CL⁻ = 125 MG/L

(2) 1000 HRS 1-16 TO 0300 HRS 1-17
COMPOSITE B.O.D. = 37 MG/L
T.S.S. = 261 MG/L
CL⁻ = 32 MG/L

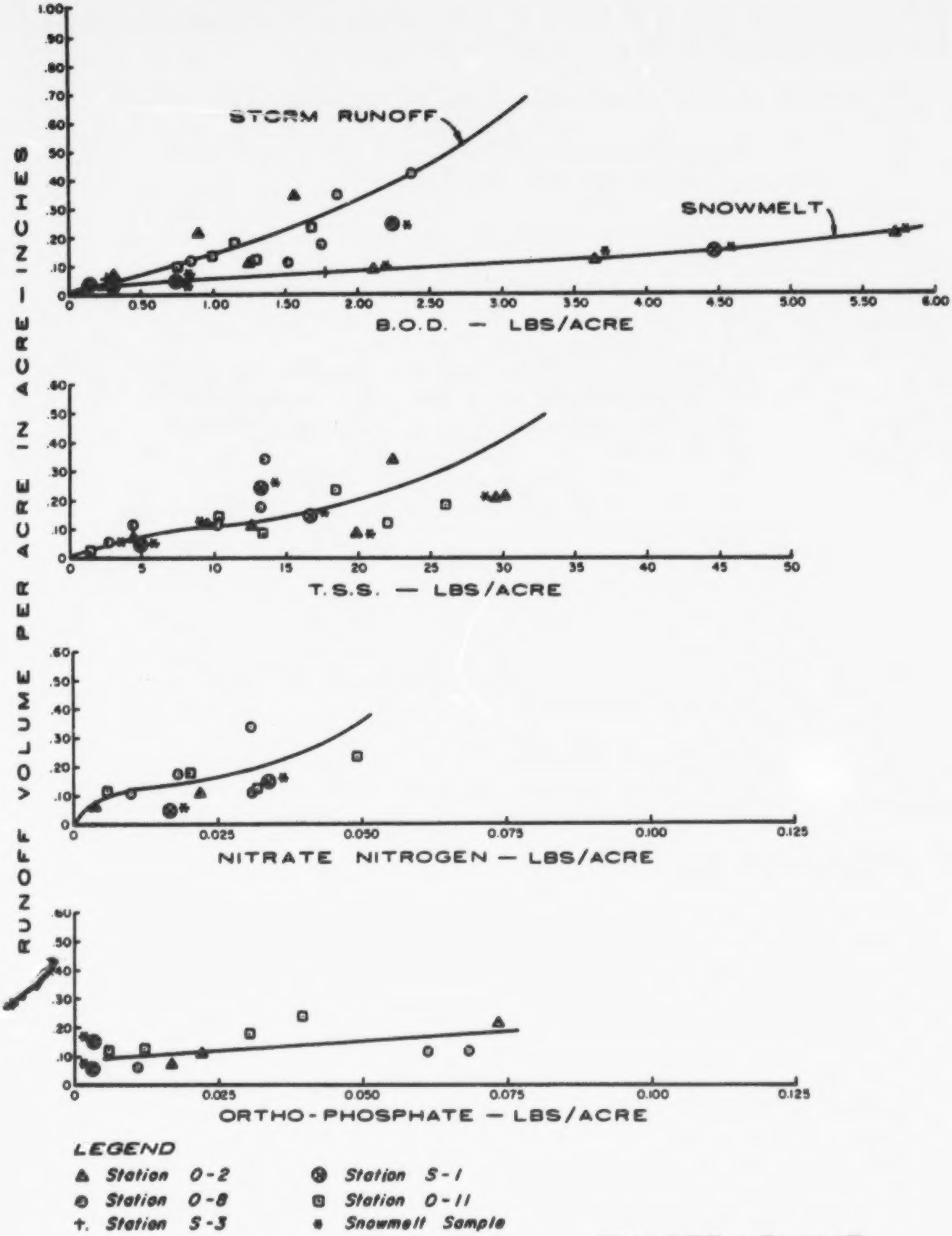
RUNOFF CHARACTERISTICS
STATION S-1
THOMPSON AVENUE STORM SEWER

FIGURE 22



VOLUMETRIC RELATIONSHIP
RAINFALL - RUNOFF

FIGURE 23



RUNOFF VOLUME
VS.
B.O.D., T.S.S., NO_3-N, & O-PO_4

FIGURE 24

particular runoff. The points plotted in Figures 16 through 22 are typical of sampling intervals during the runoff.

A cursory review of Table 7 prompts the following observations.

1. Except for Station 0-6, there is little difference in the average BOD concentrations. Overflow and runoff flows both run in the 40 to 70 mg/l range. It is believed that the higher value for Station 0-6 results from the hydraulic limitation of the sewer above that location, thus limiting diluting storm waters.

2. A greater range in values was found for suspended solids concentrations in both overflow and storm water discharges, but storm water appears to be generally higher. Volatile content is low (20 to 35 percent) indicating much of the solids are inert materials such as sand and dirt, probably from streets and yards.

3. Ammonia nitrogen concentrations were somewhat higher in overflow samples. This reflects the presence of raw sewage in these waters. Nitrite and nitrate concentrations, however, were about the same for both types of samples.

4. Phosphates were of significantly higher concentrations in the overflow samples, again probably attributable to raw sewage.

STREET DEICERS

Runoff from snow melts were analyzed for chloride and chromium content to determine the effect of street deicers. The systems for which data were obtained included S-1, S-3, 0-2, 0-8, 0-11, W-3 and W-5. The concentration of chlorides varied widely according to type of system, the amount of snow melt since application, and the quantity of flow. Inspection of the data tabulated for these areas in Appendix B will show the variation in chloride concentrations from various snow melts. Figures 18 and 19 graphically illustrate the pattern of chloride concentrations during snow melts in the S-1 and S-3 areas. The quantity of chloride in pounds also increases as the flow begins to increase, but drops off after the initial flush, thus lending support to the "first flush" theory. Reference to the data tabulated in Appendix B shows this pattern. Both composite samples and grab samples were analyzed to determine chloride quantities.

The areas monitored were primarily residential areas with some major thoroughfares, but no heavy commercial or business districts where heavy salting would be expected. On the S-1 area, it was determined that approximately 15,000 pounds of chlorides were carried off in runoffs from January 15 to March 2, 1969, which accounts for practically all snow melts. This amounted to about 1075 pounds per mile of street for the season. Considerable snow, sleet and freezing rain occurred during the last half of December and the first half of January, 1969, causing extremely heavy usage of street deicers, but no significant snow melt occurred until January 15. Although no snow melt had occurred, it was apparent from the January 15 sampling that much of the salt applied before that time was no longer on the streets.

Four snow melts from Area S-1 had the following quantities of chlorides per street mile:

January 15 & 16, 1969	(0.236 inches runoff)	84.6 lbs/mile
February 14, 1969	(0.049 inches runoff)	266.3 lbs/mile
February 5, 1969	(0.038 inches runoff)	79.4 lbs/mile
February 25, 1969	(0.149 inches runoff)	36.8 lbs/mile

Chloride concentrations of up to 2724 mg/l were measured at S-3 and 2317 mg/l at S-1, both of which occurred at quite low flows at the beginning of a snow melt on February 21, 1969. For composite samples of over 7 hours the chloride concentrations did not exceed 125 mg/l, however, and ranged to as low as 32 mg/l.

The highest concentration of chlorides found in combined sewers was in the East Side Interceptor (W-3). The concentration was 817 mg/l which is considerably less than observed in separate storm sewers.

Inquiries were made to all municipalities in the study area, the State Highway Commission and the Polk County Engineer regarding the quality of highway deicers used during the 1968-1969 winter season. Based on the amount of salt shipped to these agencies, it was estimated that the Des Moines Metro area had 9270 tons of untreated rock salt, 225 tons of rock salt treated with a rust inhibitor and 295 tons of calcium chloride applied to streets and highways this season. In terms of the chloride ion, this amounts to 5380 tons, or 11,660,000 lbs. With the exception of an early snowfall on November 10, 1968, the snow removal season ran from December 17, 1968 to early March 1969. Heaviest usage occurred during the last half of December and the first half of January due to the extreme weather conditions.

River samples collected by both the Contractor and the State Hygienic Laboratory included chloride and chromium analysis starting January 1969. These results are tabuled in Appendix C. Chlorides ran significantly higher during the winter months than in the summer. During January and February, chloride concentrations above 40 mg/l were common for the stations above Des Moines, the highest being 56 mg/l. Below Des Moines, the highest concentrations recorded were 86 and 67 mg/l at the R-6 station, and 74 mg/l at R-5. During the spring and summer months, chlorides fell to below 20 mg/l, but began to climb into the 20's as the flow dropped off in the fall.

An attempt was made to correlate the chlorides sampled in the river and storm sewer monitoring programs to that applied in the study area. A satisfactory correlation could not be made, however, and this was omitted rather than make broad speculations. Other studies have indicated that approximately half of the salt applied can be expected in the discharge. (3)

Because only a small quantity of chromium treated salt was used in the study area, this material was not included in the previous discussion. Chromium analyses were run on snow melt and river samples, and are shown in the tables in the appendices. The highest concentrations were found in the S-3 drainage area, a very small part of which received treated salt. Concentrations of 1210 and 876 ug/l were recorded from two different snow melt periods. Both of these occurred during very low flows at the beginning of the snow melt.

SECTION VII

RAINFALL-RUNOFF STUDIES

GENERAL

The original primary objective of this part of the study was to determine the relationship between rainfall and sewage flows in the combined sewer systems. A secondary objective was to develop the relationship of runoff to the amount and intensity of rainfall for various types of land use within the study area. During execution of the study it became apparent that the primary objective of relating rainfall to combined sewer flows could not be met due to a number of unanticipated conditions in the field. As a result, greater emphasis was placed on the secondary objective and the development of a procedure for predicting the quantity of pollutants carried to the stream by runoff. A rain gauging network was established for Des Moines and its environs which effectively covered the developed watersheds in the study area. Six rain gauge stations were maintained during the study in addition to the two existing Weather Bureau stations.

From the rainfall monitoring program, detailed rainfall-runoff relationships were developed for four selected watersheds. The data obtained from the supplemental monitoring, together with historical data furnished by the U. S. Weather Bureau, was useful for establishing rainfall intensity distribution with regard to total depth and duration of rainfall.

Much of the published data deals with precipitation in the magnitude of intense storms and is primarily concerned with design of drainage facilities for peak runoff flows. A number of attempts have been made to relate the effect of lesser magnitude storms to combined sewer overflow situations (4, 5, 6), but these have generally been based on assumed runoff relationships. The intense storms monitored during the study period were not of the magnitude normally considered adequate for design of storm drainage facilities, and the storm pattern frequently was not such to produce maximum runoff. They did, however, provide data for moderate storms from which coefficients of runoff were developed. From these , evaluation of the annual or seasonal pollution contributions of combined or separate storm runoff is possible. Where local conditions dictate treatment of combined or separate urban runoff, information of this type is desirable for design as well as for evaluating the degree of treatment that will be provided by a given system or facility.

The original concept was to provide a rain gauging network which would cover all developed sanitary and combined sewer watersheds in the Des

Moines metropolitan area. Both combined sewer systems and separate sanitary systems are known to carry large quantities of extraneous water during wet weather. It was desired to determine the extent and source of such flow and the influence of direct rainfall runoff in the individual systems. This, then, was the initial basis for the flow monitoring system as well as the rainfall monitoring.

In the actual execution of the monitoring programs, extraneous flows were found to be of a completely different nature than anticipated. Very high infiltration overloaded separate sanitary systems and produced a prolonged surcharge in all major trunk sewers. An extended period of high river stage further complicated hydraulic measurements, negating attempts to directly measure runoff through the major riverfront combined systems. As a result, it was concluded that factors other than direct rainfall runoff were of such magnitude as to make any attempt to correlate rainfall and sanitary flow very difficult, if not impossible. Therefore, rainfall-runoff correlations were limited to separate storm sewer systems.

DESCRIPTION OF RAIN GAUGE STATIONS

The rain gauge stations are located as shown on Figures 14 and 25 and are described as follows:

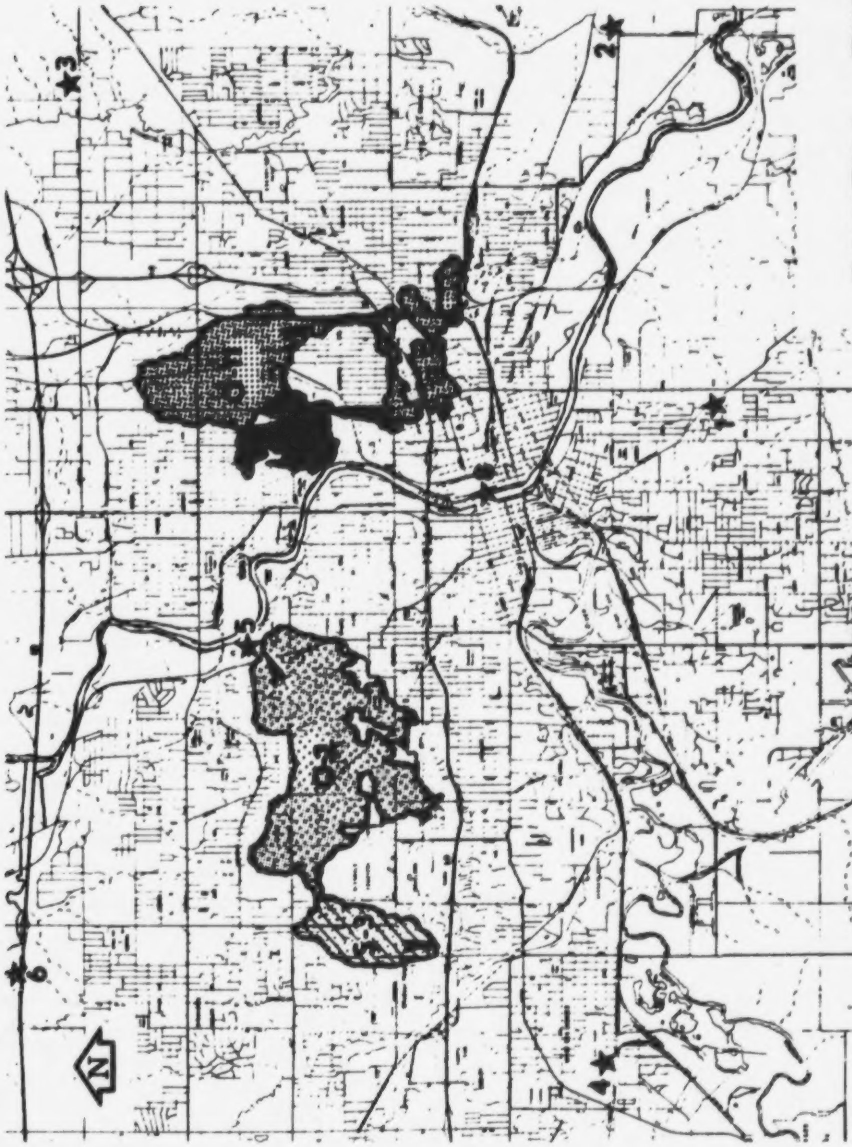
Station No. 1 is located one-half block west of Indianola Avenue on the north side of Park Avenue. The site is the location of a water standpipe operated by the Des Moines Waterworks and was large enough that the rain gauge could be located so as not to be influenced by the standpipe.

Station No. 2 is located on the premises of the Phillips Petroleum Company, 4400 Vandalia Road. The site is used for offices, warehouse and bulk storage of petroleum products and provided ample open space for locating the rain gauge.

Station No. 3 is located on the premises of the KSO Radio Broadcasting Company at 3900 N. E. Broadway. A fenced swimming pool which is no longer used provided a secure area for the gauge. The diving board platform served as an excellent base for the rain gauge, as was shown in Figure 12.

Station No. 4 is located in the West Des Moines maintenance yard at 1405 Maple Street. The site is used for equipment storage and maintenance and adequate space was available to prevent interference with the rain gauge.

Station No. 5 is located in the vehicle parking compound of the Iowa National Guard Armory at 1915 Prospect Road.



RAINFALL - RUNOFF STUDY AREAS

★ *Raingage Location*

FIGURE 25

Station No. 6 is located at the Urbandale Sanitary District Wastewater Treatment Plant. The site is immediately north of Interstate 35-80 and one-half mile east of 72nd Street.

Station No. 7 is an existing Weather Bureau gauge located at the Municipal Airport. This is a first order station, for which precipitation records date to 1877. Hourly precipitation records from this station are published monthly.

Station No. 8 is an existing Weather Bureau Station located at the Federal Court Building between Walnut and Court on East First Street. Data from this station is not published, but copies of the gauge charts were obtained for the storms studied.

All of the sites were fenced areas and no vandalism problems were encountered. Arrangements were made with each owner to obtain a key to the premises or to maintain the gauges when the premises were open.

STUDY AREAS

Four watersheds were selected for making detailed analyses of rainfall and runoff. Typical areas were selected to give various topographic features, land use and degree of development. The watersheds required relatively well defined boundaries, and could not be interconnected with other systems. The basin also required an outlet at a single point, and needed to be reasonably accessible and not subject to backwater or complex hydraulic situations.

It was desired to monitor a heavy commercial area in the business district, but because of conditions at the outlet, interconnection with other systems, and the lack of well defined boundaries, it was not possible to locate a suitable area.

The areas which were selected for detailed study are described below and are shown on Figure 25.

S-1 - Thompson Avenue Storm Sewer

This is a separate storm sewer serving an area of 315 acres. The area is primarily medium density residential with an estimated population density of 12 people per acre. The topography is rolling with an elevation of about 796 feet near the flow monitoring point . The Thompson Avenue storm sewer is a 5'-3" wide by 3'-10" high concrete box at the outlet, discharging into the Birdland Park Boat Marina on the east bank of the Des Moines River. This outlet was submerged during high river stage, as shown in Figure 62, but several storms were monitored before and after this period.

There are several areas adjacent to and above this system which are served by combined sewer systems. It could only be assumed that sufficient inlet capacity existed in the combined sewer areas to prevent overland flows from these areas into the area being monitored. While it is possible that this was not always the case, it is felt that the error introduced from this source would be small.

The approximate time of concentration from the most distant point to the flow monitoring point is 30 minutes. For design, a composite coefficient of runoff "C" of approximately 0.58 would be used for the area as a whole.

Flow measurement was accomplished by a sharp-edged rectangular weir without end contractions. Water levels were recorded by the bubbler-type installation described in Section V.

S-3 - Cummins Parkway Storm Sewer

This separate storm sewer system serves a relatively new residential area of 356 acres. The area is medium density residential with an estimated density of 12 people per acre. The topography is rolling with an elevation of about 838 feet at the flow monitoring point to about 975 feet at the furthest upstream point. The area is approximately 8200 feet in length and 3100 feet in width at the widest point. The time of concentration from the furthest point to the point of flow monitoring is estimated to be 40 minutes for a design storm. A composite coefficient of runoff "C" of approximately 0.53 would be used for determining design storm runoff.

The monitoring point was a 6-foot wide by 8-foot high concrete box culvert at Windsor Drive and Cummings Parkway. The drainage from this system flows in an open ditch from approximately 1700 feet above the monitoring station to a point near 63rd and Grand where it discharges to Walnut Creek. Flow measurement was accomplished by a sharp-edged rectangular weir without end contractions and recorded by the bubbler-type recorder. The installation is shown in Figure 10.

0-2 - Closes Creek Watershed

This watershed of 1673 acres is drained by numerous storm sewer systems with 107 acres served by combined sewers. The area is medium density residential with light commercial in neighborhood shopping centers. The population density for the entire area is 7.4 people per acre, but varies considerably with large areas having a density of 12 people per acre or greater. Topography varies from a relatively flat plateau at about elevation 970 to very steep, wooded ravines along

Closes Creek. The flow monitoring point is just above the Des Moines River near Harding and Prospect Roads and is at elevation 797. The distance of flow travel from the monitoring point to the furthest point is about 20, 000 feet and the width of the area is about 7, 000 feet at the widest point. The time of concentration from the monitoring point to the furthest point is estimated to be 70 minutes for a design storm. The composite coefficient of runoff "C" for the area is estimated to be 0. 57 for the design storm.

Flow measurement was by stage-discharge relationship with an excellent point of permanent control. The control section was a concrete encased sewer crossing the creek and is shown in Figure 10. Sufficient low flow measurements were made to establish the lower section of the rating curve and high water measurements were made as often as possible to provide a basis for projecting the curve. Water levels were recorded by a bubbler-type recorder. Only during very high river stage was the control effected by backwater.

Because of the combined sewer areas, the flow measured at the 0-2 station was adjusted for the storm water flow in the Closes Creek trunk sewer. Numerous overflows exist in the combined system which bypass during wet weather. Measured flow was increased on a prorated basis of the total watershed to the separately served area, but the increase was limited to the reserve storm water capacity in the main trunk.

0-11 - The 20th Street Storm Sewer

This system serves an area of approximately 1, 166 acres, but is broken up by several independent separate and combined systems as shown in Figure 14. The system was originally thought to receive overflows from the sanitary system, thus the "0" designation. Extensive field investigation failed to locate any overflows and it is now considered a separate system.

The area is residential with extensive industrial and commercial development. The topography of the area is relatively flat, sloping from an elevation of 840 at the furthest upstream point to an elevation of 794 at the monitoring point with a flow travel distance of 19, 500 feet. The time of concentration from the furthest point to the monitoring point is estimated to be about 85 minutes for a design storm. The composite coefficient of runoff "C" for a design storm is estimated to be 0. 53 for the entire area, but would vary considerably within the area because of large areas of both dense industrial and commercial development and vacant land.

The outlet of this system is at the northern-most end of Dean Lake, and is a 4-foot wide by 6-foot high concrete box sewer at that point. Flow was measured by a sharp-edged rectangular weir without end contractions and the water level was recorded by a bubbler type recorder. The installation is shown in Figure 7.

RAINFALL-RUNOFF RELATIONSHIPS

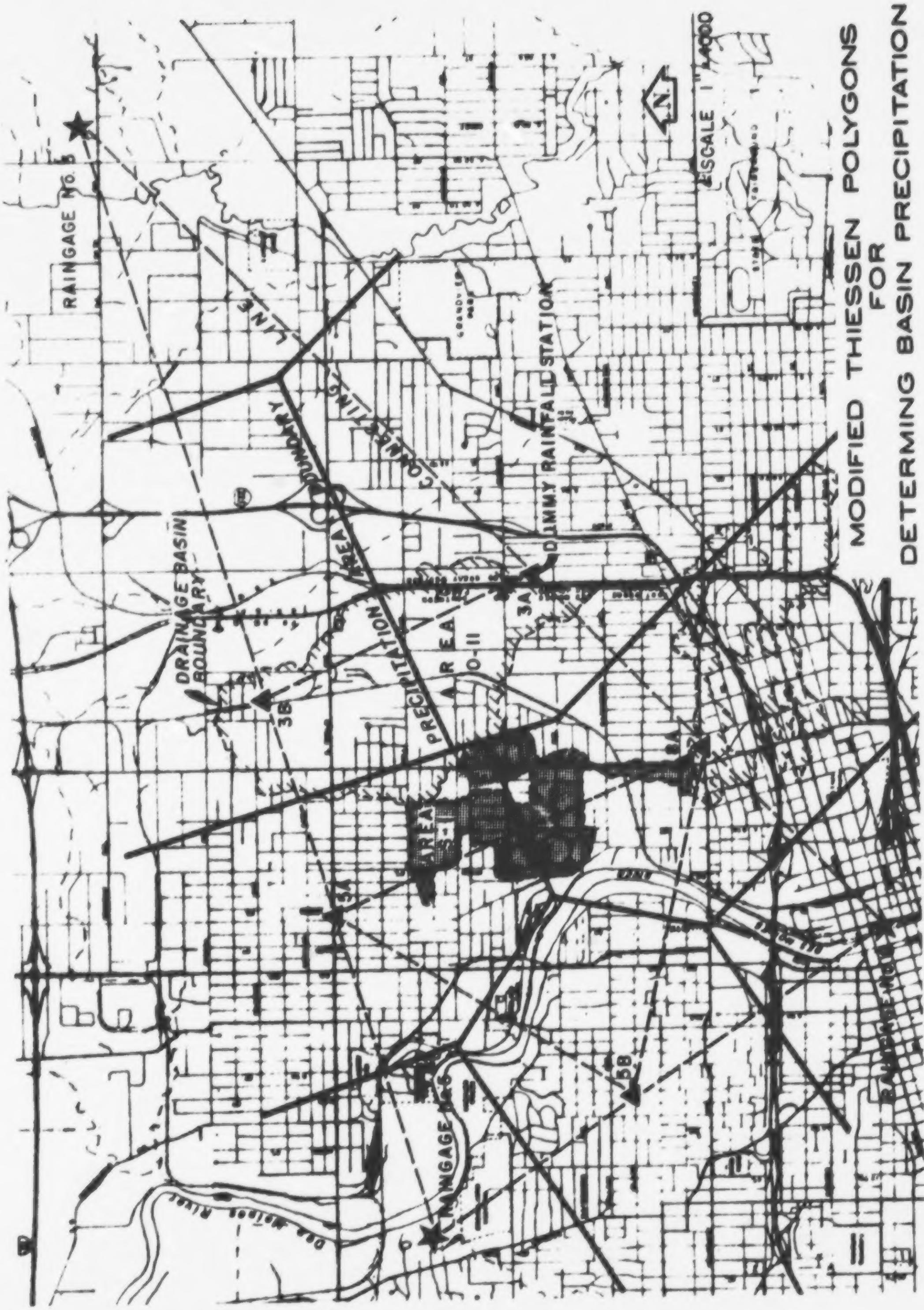
Determination of Rainfall

A method for determining the amount and intensity of rainfall on each area was developed, adapting the Thiessen Method to analysis by electronic computer. The step by step procedure used in setting up these analyses is described below.

1. Watershed boundaries for each area were outlined in 1" = 400' scale topographic maps. City sewer plats were checked for storm sewer locations, and drainage as well as storm sewer and inlet locations were field checked.

2. Rain gauges were dispersed over the entire study area due to the desire at the outset of the study to correlate rainfall to wet weather sanitary flows in all of the major basins in the collection system. As a result, the gauges did not fall within the separate storm drainage basins actually monitored. Had Thiessen Polygons been drawn on ly around actual rain gauging stations, the rainfall calculated for a given basin would generally have been based on only one of the rain gauge measurements. A procedure was developed by the contractor to handle this condition, which is similar to the procedure used by Myers (7) for determining average rainfall on remote mountainous areas. Intermediate points, referred to herein as dummy rainfall stations, were established on a connecting line between rain gauges. This permitted interpolation of rainfall to points in or near the area being monitored. These points, the dummy rainfall stations, were then used to develop a system of Thiessen Polygons which would encompass the basins monitored. This procedure is illustrated in Figure 26.

   This procedure varied from standard practice; therefore certain limitations were placed on the data used for analysis. In order to assure that interpolation between rain gauges was valid, the rain gauge readings


RAINGAGE NO. 5
DRAINAGE BASIN BOUNDARY
PRECIPITATION AREA BOUNDARY
CONNECTING LINE
DUMMY RAINFALL STATION
AREA O-11
AREA S-1
3A
3B
5A
5B
8A
RAINGAGE NO. 6
Des Moines River
N
SCALE 1" = 4000'

MODIFIED THIESSEN POLYGONS FOR DETERMING BASIN PRECIPITATION

FIGURE 26

used for calculating rainfall on each of the monitored areas was carefully checked to see that rainfall was occurring relatively uniformly at each station. This procedure is explained in subsequent paragraphs.

3. The watershed within each polygon was planimetered to determine the area in acres. Since rainfall at dummy stations is a function of that recorded at the gauges, rainfall to the entire watershed was equated to the rainfall observed at the three gauges nearest the area.

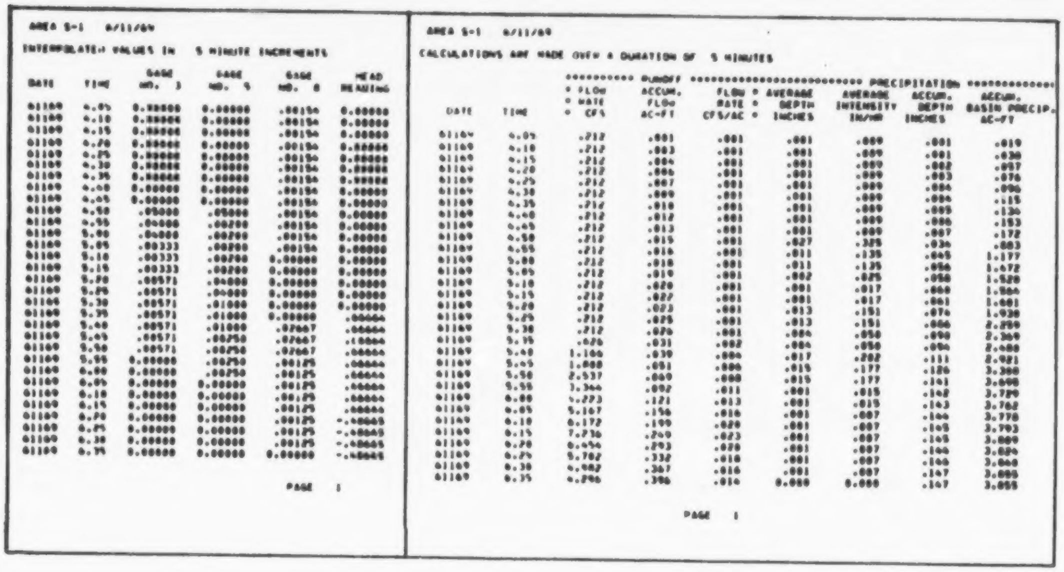
Procedure for Analysis

The procedure used for developing rainfall and runoff relationships is as follows:

1. A computer program was developed to process rainfall and runoff measurements into 5 minute increments. Input data consisted of time-depth values describing each rainfall and head values for the flow measuring device, each in 5 minute multiples of the clock hour. Also, for each watershed the following constants were input: head correction for flow chart readings, watershed area in acres, the estimated minimum time of concentration for runoff, the equation for determination of discharge rate, and an equation for determination of the volume of rainfall on the watershed.

2. A sample of the computer print-out is shown in Figure 27. For each 5 minute increment throughout the storm and runoff period, the following information is given:

For Runoff	- Flow rate (discharge) in cfs. - Accumulated runoff since the beginning of the storm, in acre-feet. - Rate of flow per acre of watershed, in cfs/acre.
For Rainfall	- Average incremental depth of precipitation over entire watershed for 5-minute increment, in inches. - Average incremental intensity of precipitation over entire watershed for 5-minute increment, in inches/hour.



```
AREA S-1   6/11/69
INTERPOLATED VALUES IN   5 MINUTE INCREMENTS

                           GAGE        GAGE        GAGE        HEAD
DATE       TIME          NO. 3       NO. 5       NO. 8      READING

                                                        PAGE   1
```

```
AREA S-1   6/11/69
CALCULATIONS ARE MADE OVER A DURATION OF   5 MINUTES

                    ********** RUNOFF *********************** PRECIPITATION ************
                    * FLOW    ACCUM.     FLOW * AVERAGE    AVERAGE    ACCUM.     ACCUM.
                    * RATE     FLOW      RATE *   DEPTH  INTENSITY    DEPTH  BASIN PRECIP.
  DATE     TIME     *  CFS    AC-FT   CFS/AC *  INCHES     IN/HR    INCHES     AC-FT

                                        PAGE   1
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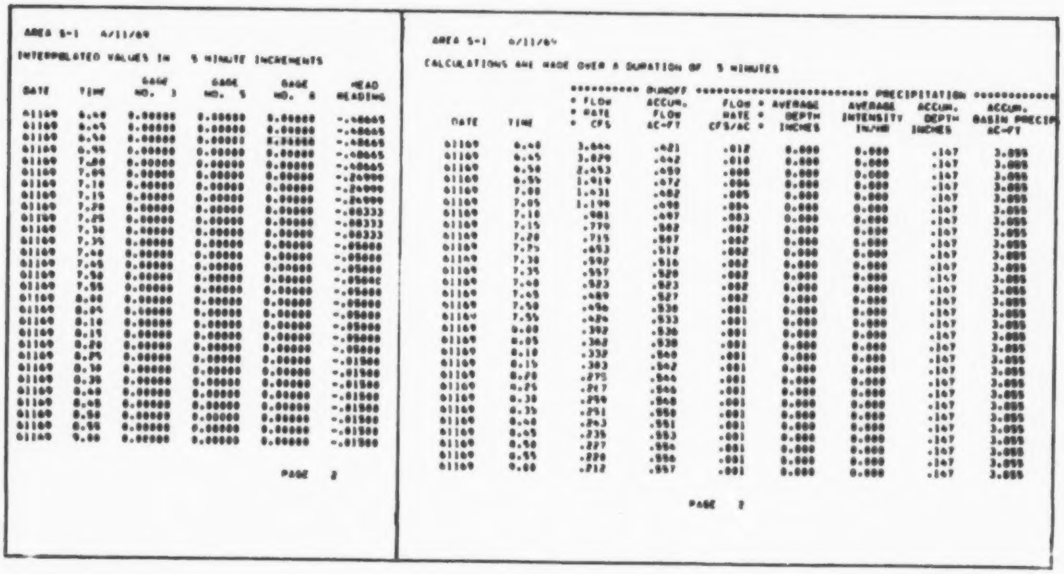

```
AREA S-1   6/11/69
INTERPOLATED VALUES IN   5 MINUTE INCREMENTS

                           GAGE        GAGE        GAGE        HEAD
DATE       TIME          NO. 3       NO. 5       NO. 8      READING

                                                        PAGE   2
```

```
AREA S-1   6/11/69
CALCULATIONS ARE MADE OVER A DURATION OF   5 MINUTES

                    ********** RUNOFF *********************** PRECIPITATION ************
                    * FLOW    ACCUM.     FLOW * AVERAGE    AVERAGE    ACCUM.     ACCUM.
                    * RATE     FLOW      RATE *   DEPTH  INTENSITY    DEPTH  BASIN PRECIP.
  DATE     TIME     *  CFS    AC-FT   CFS/AC *  INCHES     IN/HR    INCHES     AC-FT

                                        PAGE   2
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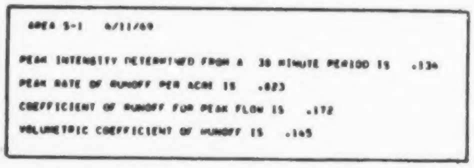


```
AREA S-1   6/11/69

PEAK INTENSITY DETERMINED FROM A  30 MINUTE PERIOD IS   .134

PEAK RATE OF RUNOFF PER ACRE IS   .023

COEFFICIENT OF RUNOFF FOR PEAK FLOW IS   .172

VOLUMETRIC COEFFICIENT OF RUNOFF IS   .145
```

COMPUTER PRINT-OUT
FOR
RAINFALL - RUNOFF ANALYSIS

FIGURE 27

- Accumulated depth of precipitation on watershed since beginning of storm, in inches.
- Accumulated precipitation on watershed, in acre-feet.

Also, the following items are summarized at the end of each computer run:

- Maximum average rainfall intensity, in inches/hour, occurring in time of concentration given (i.e., the greatest value obtained by averaging consecutive incremental intensities for a period equal to the time of concentration).
- Peak rate of runoff per acre observed during the runoff period, in cfs/acre.
- Coefficient of runoff for peak flow, C, which is 1.008 times the peak rate of runoff per acre divided by the maximum rainfall intensity for the concentration period.
- Volumetric coefficient of runoff, C_V, which is the acre-feet of accumulated runoff divided by the acre-feet of accumulated precipitation.

3. Storms were selected for analysis for each watershed, generally using the largest magnitude storms for which reliable data was available. Also, a few moderate storms were selected to give a range of values for developing a curve.

   For each storm rainfall and flow data was picked from the respective chart trace, tabulating time of day and chart reading for each break point in the trace. Time values were in multiples of 5 minutes and were recorded from the beginning of each storm until runoff ceased or returned to near base line flows. The tabulated time-depth values were then key punched on cards for computer analysis.

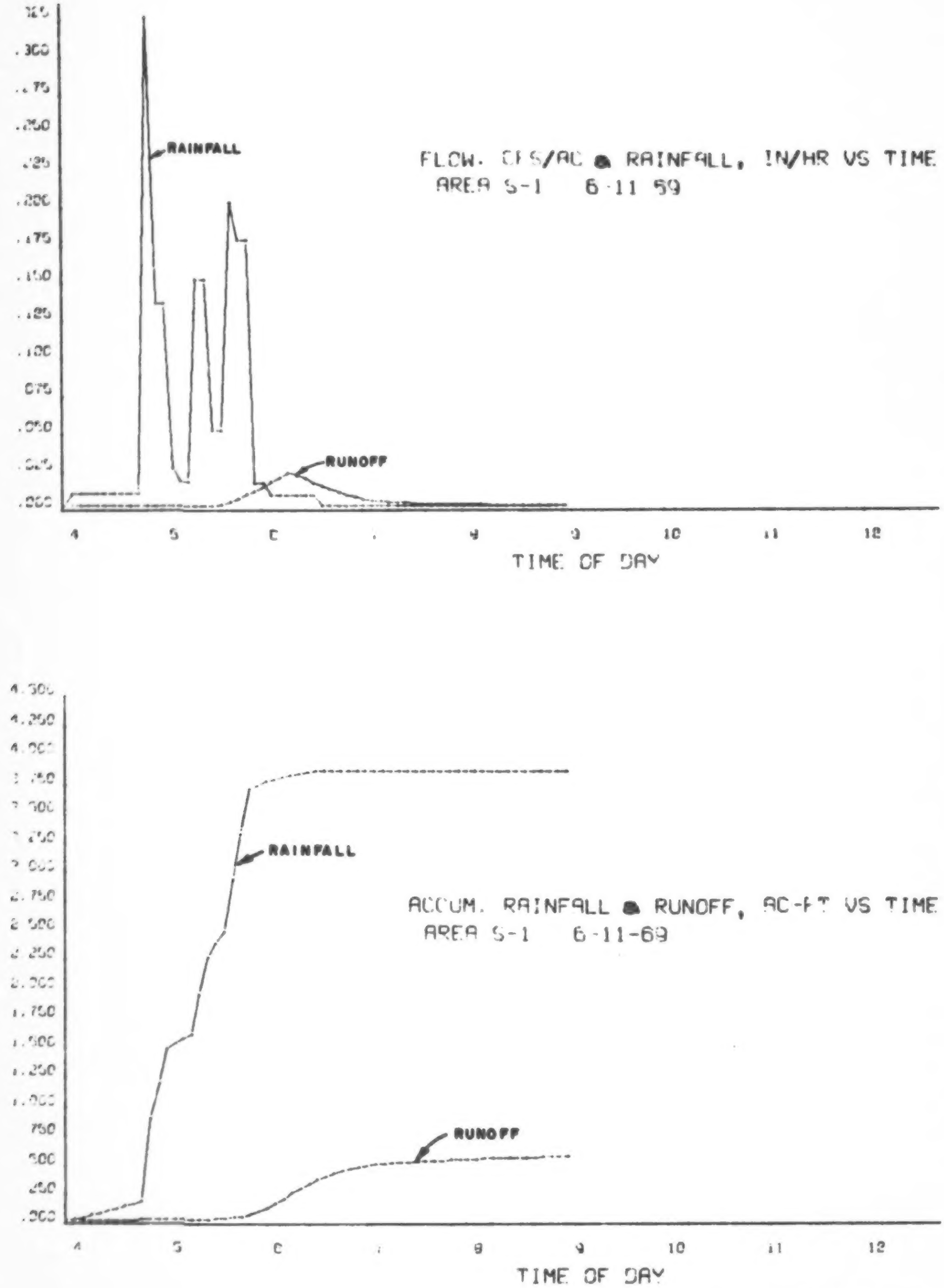
4. Rainfall and runoff values were first interpolated to 5-minute increments by the computer, based on the time-depth values selected from break points on the chart trace. The analysis was then run to obtain the output data previously described. To assist in the review of

the print-out information, two graphs were plotted by the computer. An example of these are shown for each of the four areas studied in Figures 28 to 31. The upper part of the figure is a plot of rainfall intensity and rate of runoff per acre against time of day. Since the conversion from inches per hour to cfs per acre is for practical purposes unity, the ratio that the runoff per acre is to the rainfall intensity is the coefficient of runoff "C" in the Rational formula. The lower part shows accumulated rainfall and accumulated runoff against time of day. These show at a glance the pattern of rainfall as compared to the runoff from the storm.

5. The computer analysis of each storm was reviewed in detail to insure reliable results. First, the interpolated values were reviewed to see that rainfall actually occurred at the gauges used in the analysis and that it was reasonably consistent and of approximately the same magnitude throughout the area. Rainfall for the three gauges effecting such area were reviewed simultaneously from computer print-outs giving rainfall in 5-minute increments. Scattered showers and thunderstorms, indicated by very erratic rainfall between stations, were omitted from further analysis. The elimination of these localized erratic rainfalls is not considered to have biased the data since they often were of a duration less than the time of concentration to the monitoring point and would not have been used for analysis anyway.

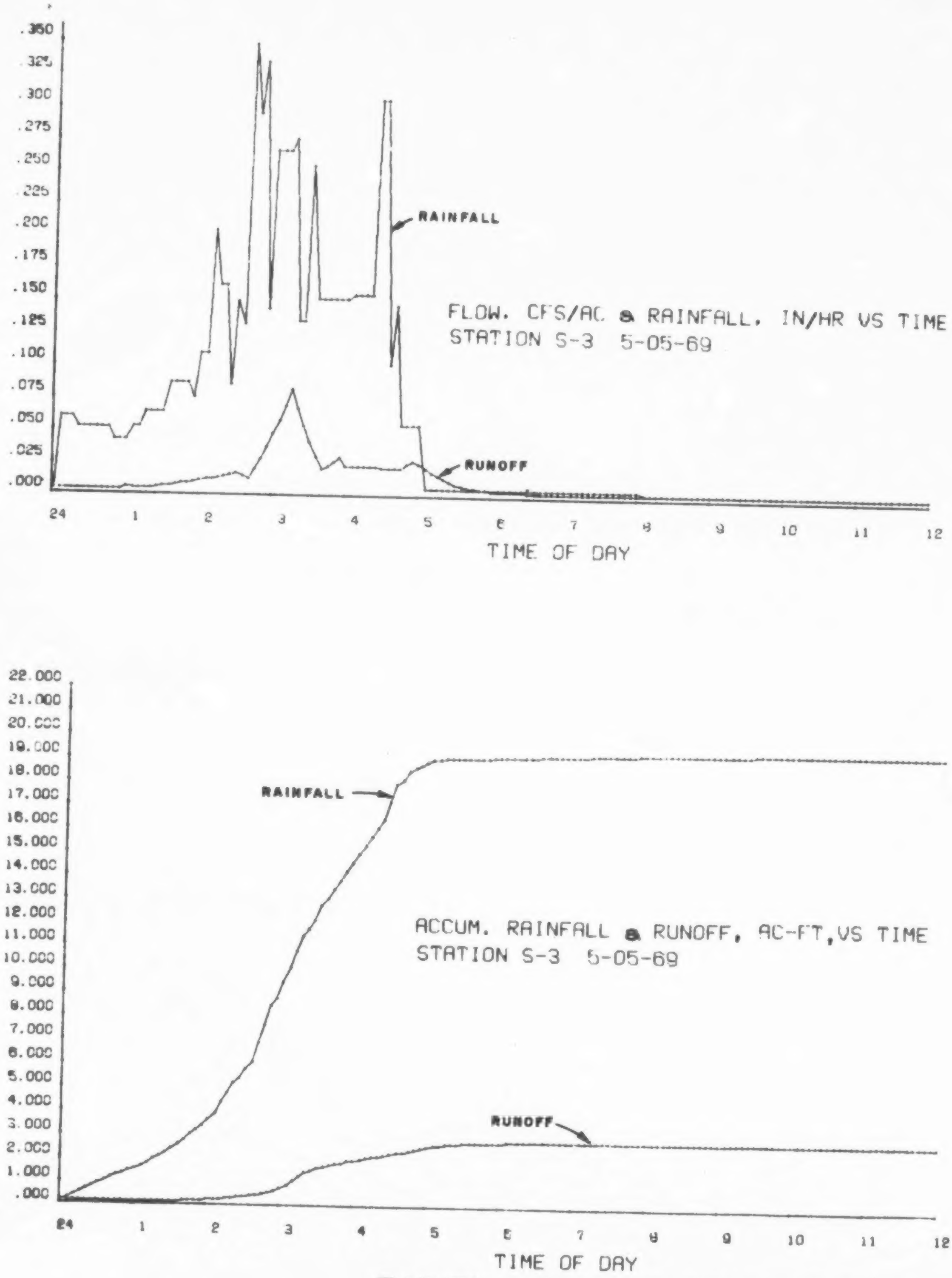
   Where base flows existed, these were deducted for further analysis. At the Closes Creek monitoring station, the flow was also adjusted for estimated storm flow in the combined sewer. The duration of the storm and the runoff was noted, as was the total accumulated depth of precipitation. Values for the volumetric coefficient of runoff, "C_V" were then manually adjusted to account for base flows and any other irregular conditions.

   Where the coefficient of runoff for peak flow "C" was calculated, it was checked to see that the storm continued through the time of the peak flow. The time of concentration was adjusted as appropriate for each storm, and new values for the average intensity were computed. If the storm did not continue through the



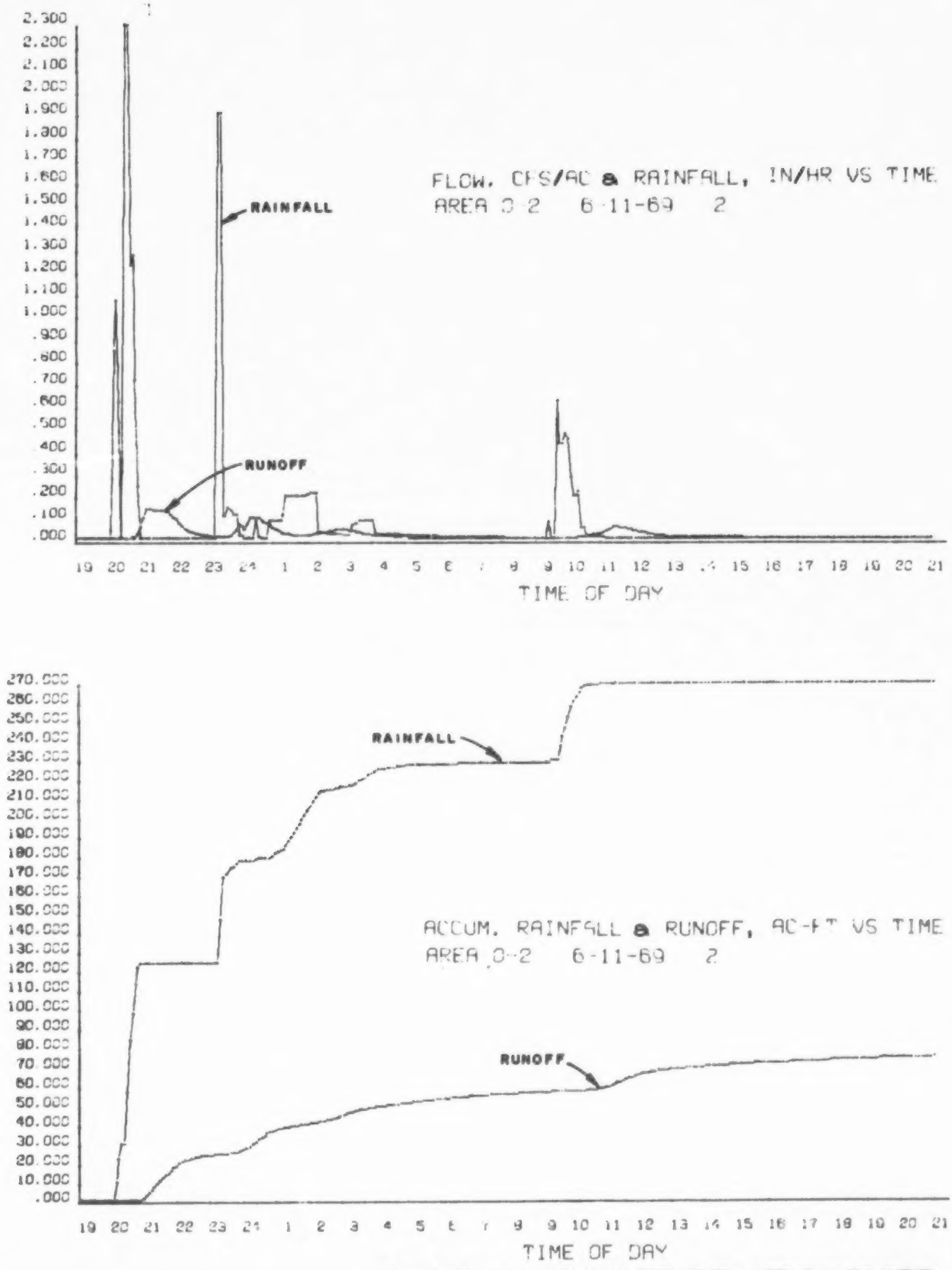
RAINFALL - RUNOFF RELATIONSHIPS
THOMPSON AVE. STORM SEWER

FIGURE 28



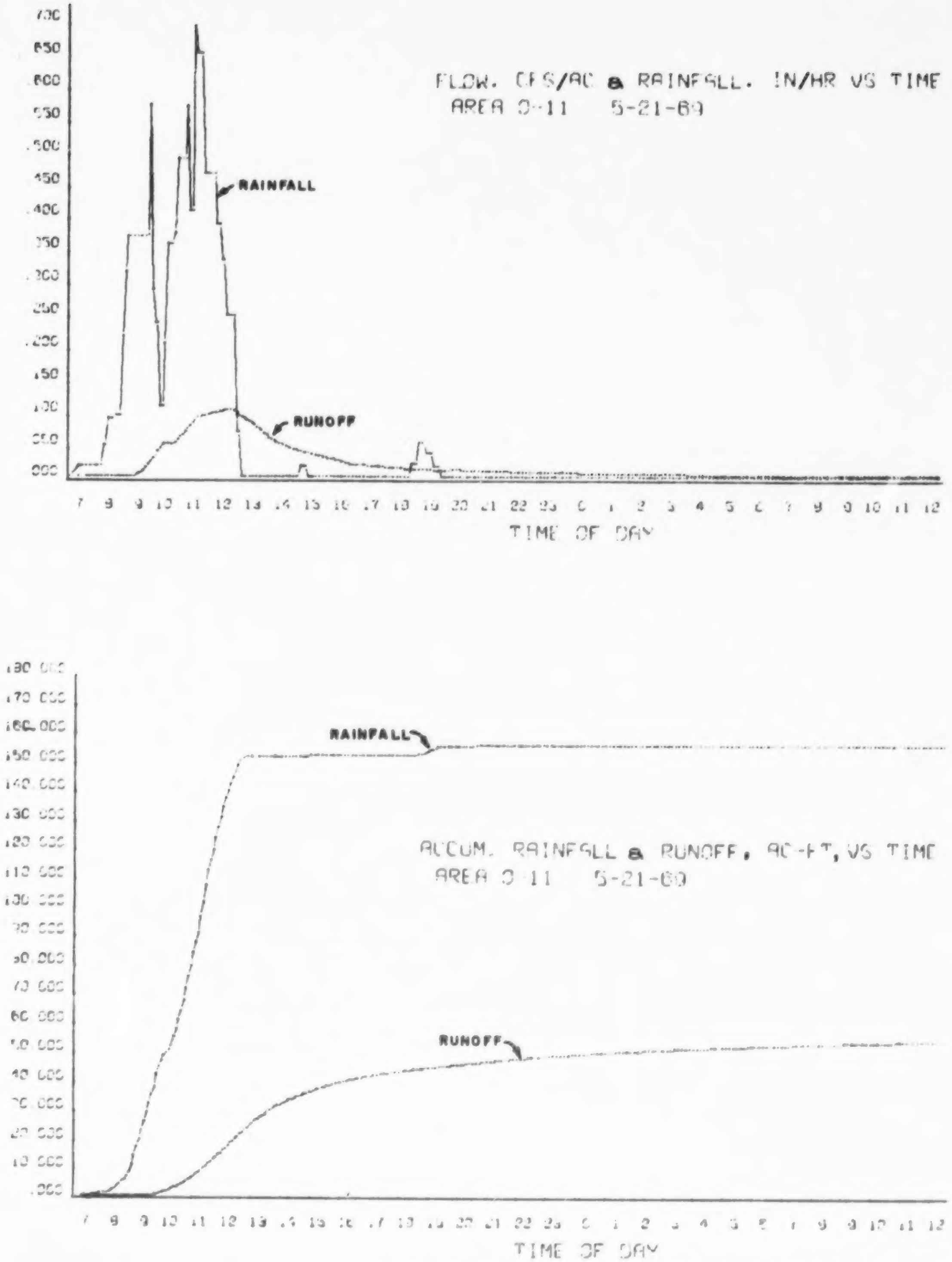
RAINFALL-RUNOFF RELATIONSHIPS
CUMMINS PARKWAY STORM SEWER

FIGURE 29



RAINFALL - RUNOFF RELATIONSHIPS
CLOSES CREEK

FIGURE 30



RAINFALL - RUNOFF RELATIONSHIPS
20TH STREET STORM SEWER

FIGURE 31

peak runoff, the value "C" was not computed. Figures 28 and 30 show storms that did not continue through the peak runoff, whereas Figures 29 and 31 show runoffs that peaked while the storm was still occurring.

Rainfall-Runoff Relationships

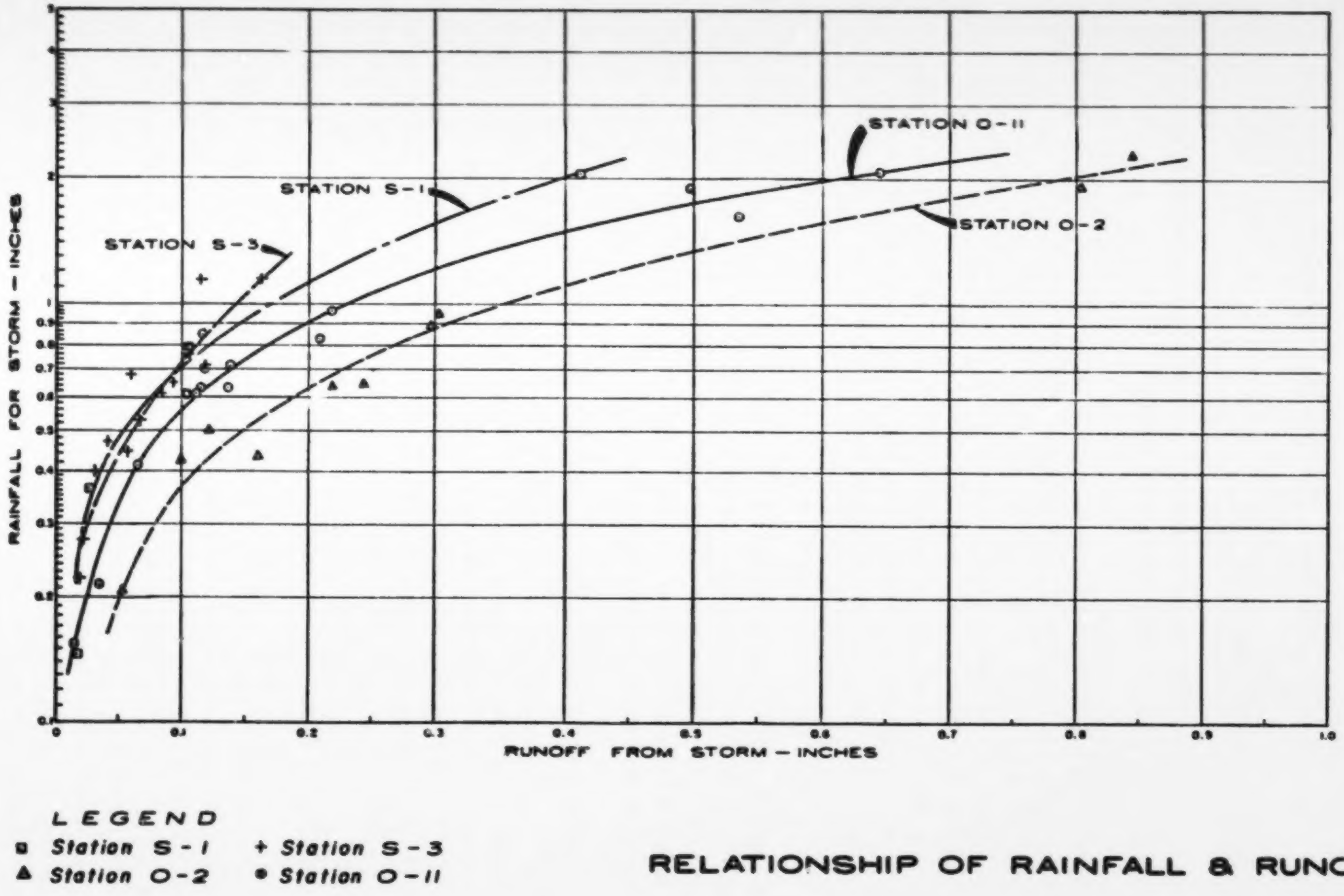
A series of curves were drawn depicting the relationship of rainfall to runoff as the depth and intensity varies.

One such set of curves is shown in Figure 32, in which the total rainfall for the storm is plotted against the total runoff. A separate curve is plotted for each of the areas to show the variation in runoff characteristics. They have been plotted on a semi-log scale to spread the points in the lower section of the curve. Plotted to a linear scale, the curves would straighten considerably. All of the areas follow the same general curve, the difference being in the degree of runoff. At any point on the curve, the volumetric coefficient of runoff, "C_v", is the funoff divided by the rainfall.

Rainfall-runoff data is plotted separately for areas 0-2, 0-11 and S-3 in Figures 33 and 34. These curves show the influence of the total rainfall depth on the volumetric coefficient of runoff, "C_v", and provide a comparison of the individual areas. Area S-1 did not provide enough values to draw a meaningful curve of this type.

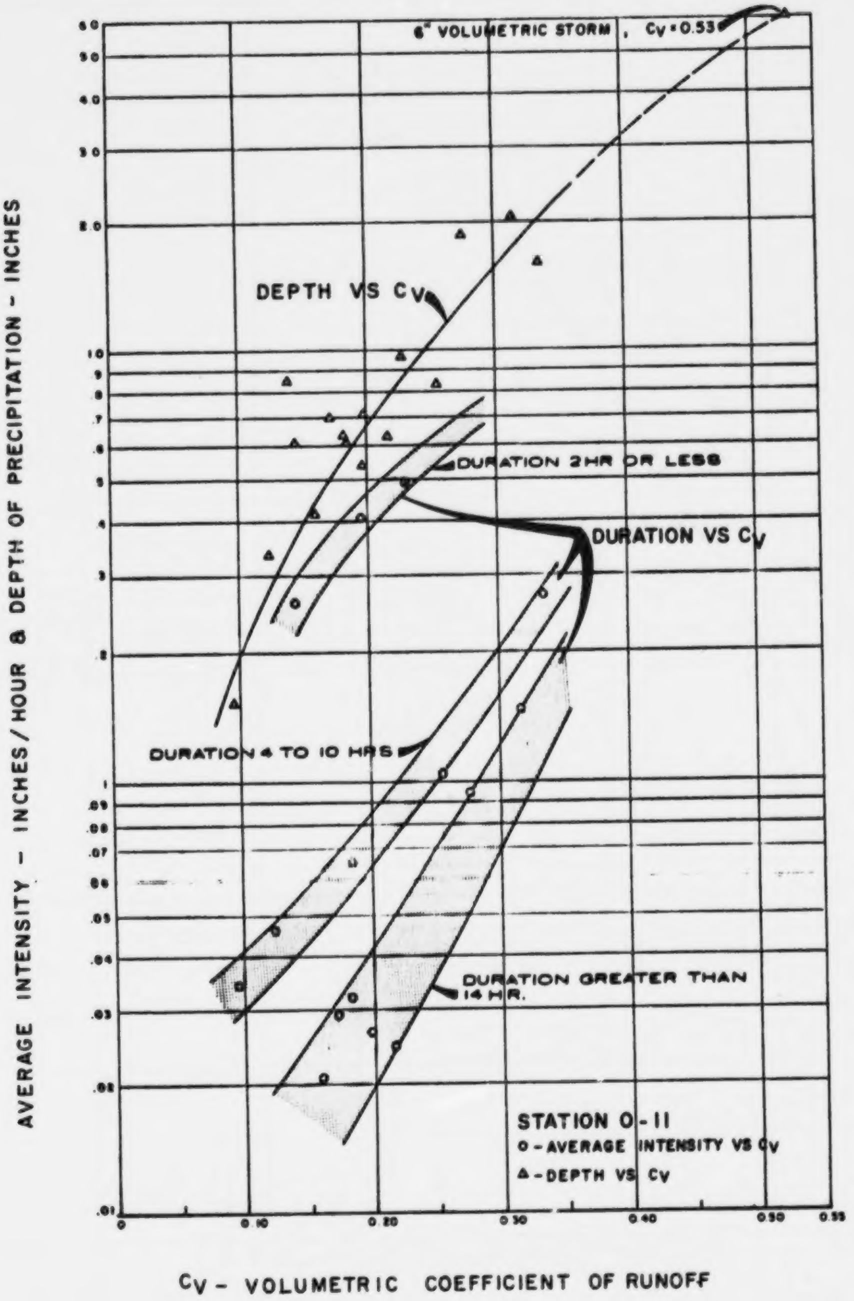
The greatest quantity of data was available for the 20th Street Storm Sewer 0-11, shown in Figure 33. The total depth of rainfall is plotted against "C_v" in the upper curve. Although the points are scattered, the general trend of the curve can be determined. The assumed value of "C_v" for a 6-inch volumetric storm is also plotted and the curve extended to that point. The curve is drawn through the grouping of values toward the right. It is assumed that values plotting substantially to the left or above the curve were from rainfalls which did not produce optimum runoff due to the storm pattern.

In this figure, "C_v" is also plotted against the average intensity over the duration of the storm. These values are widely scattered and therefore the general ranges of duration are shown. It is of interest to note that "C_v", or the percentage of runoff, increased with greater duration. Obviously, the duration is not the influencing factor. For the types of runoff measured from this watershed, the results indicate that it is the greater depth of rainfall which occurs over an extended period that produces a higher percentage volume of runoff, rather than the intensity of the storm. This was verified by inspection of the individual values plotted. In different terrain with permeable soils or greater vegetation, the results may be quite different. Also, the average intensity



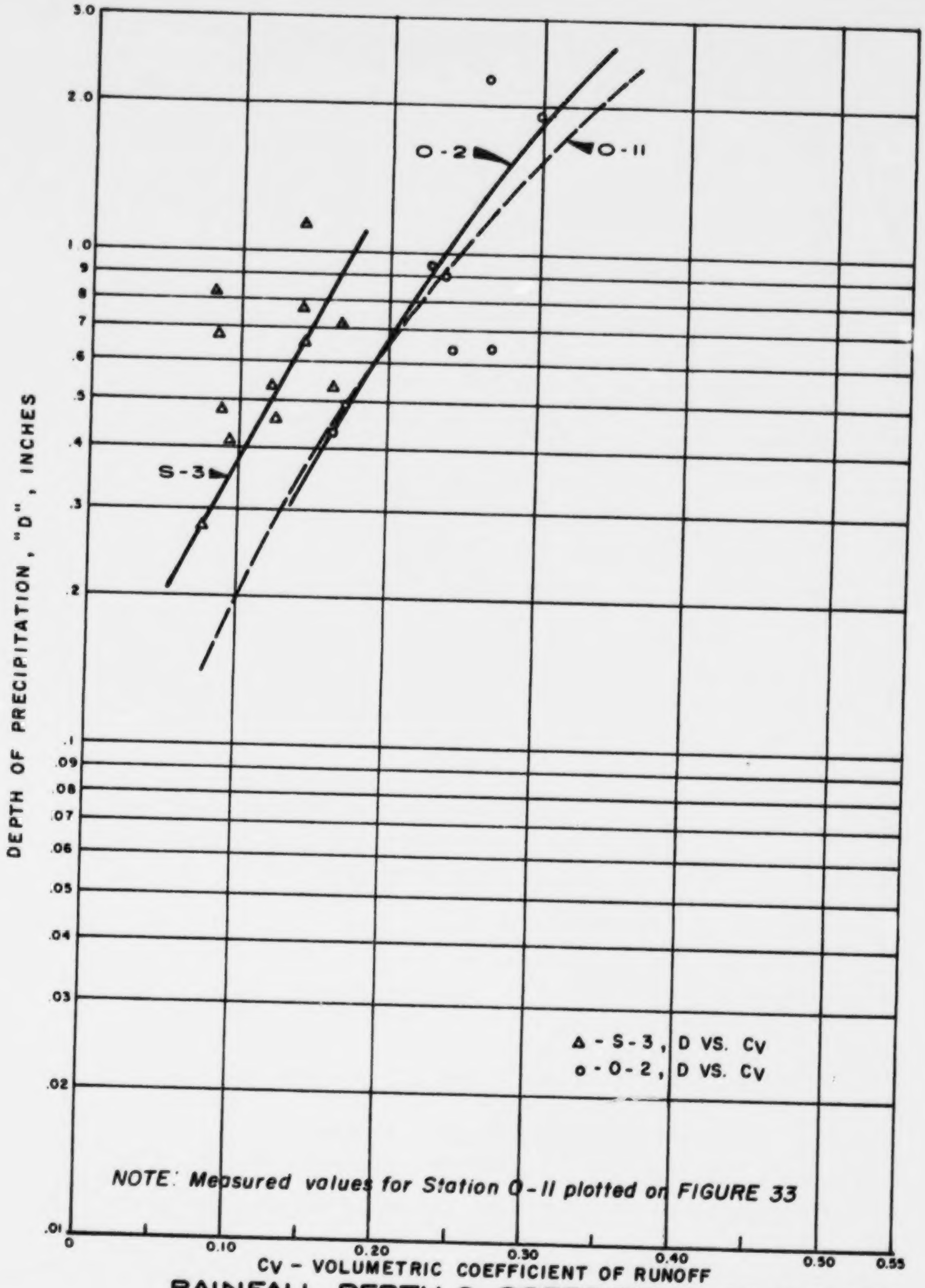
RELATIONSHIP OF RAINFALL & RUNOFF

FIGURE 32



EFFECT OF RAINFALL DEPTH & INTENSITY
ON
COEFFICIENT OF RUNOFF

FIGURE 33



NOTE: Measured values for Station O-11 plotted on FIGURE 33

RAINFALL DEPTH & COEFFICIENT OF RUNOFF

FIGURE 34

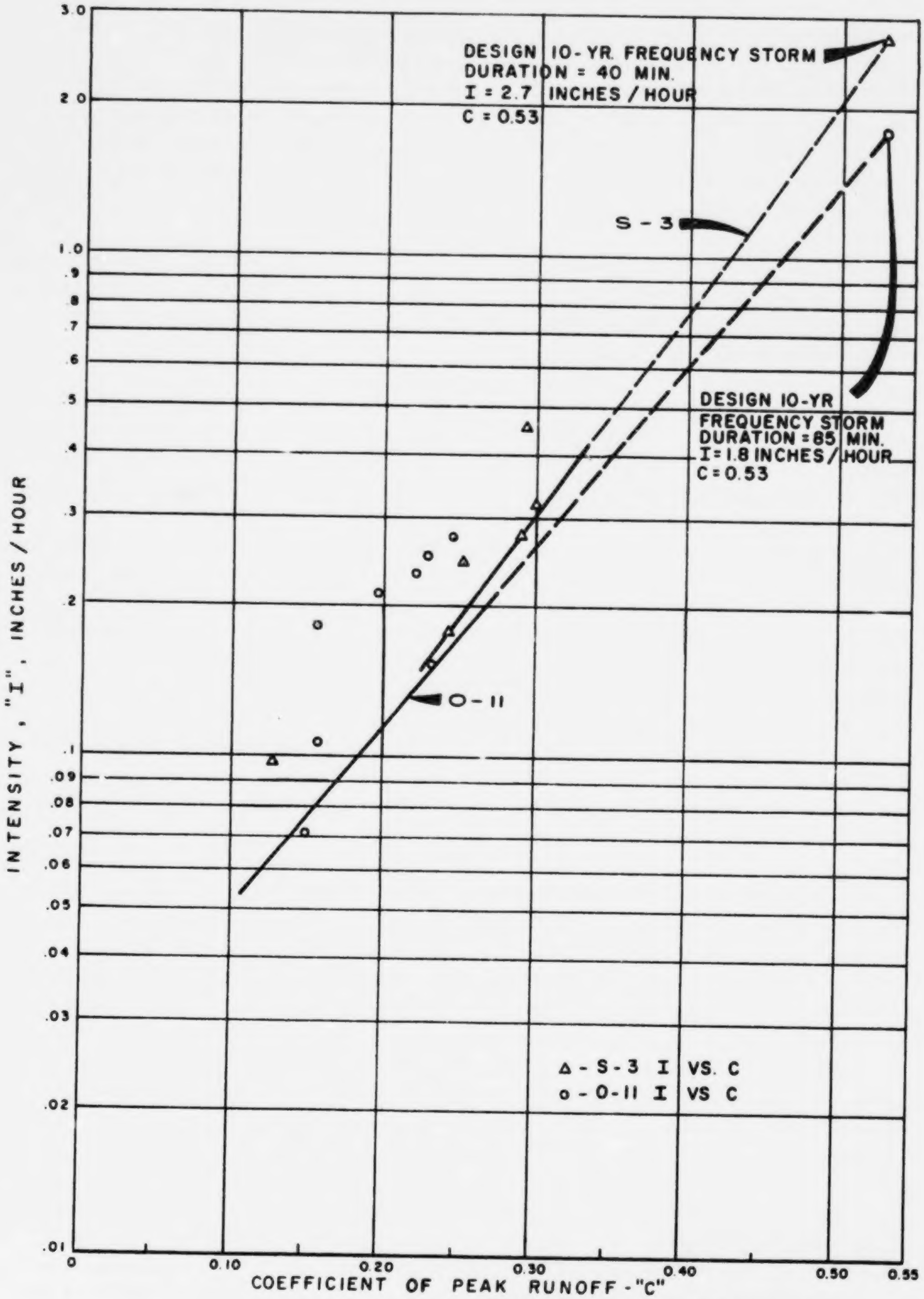
is a rather arbitrary figure since the observed intensity will most likely vary greatly during the rainfall period. However, it is obvious from this, that the average intensity or the duration alone are not adequate parameters for estimating the total volume of runoff from a storm, although they may very well be a judgment factor. The depth of rainfall appears to provide a better basis for estimating the total quantity of runoff.

Time and resources limited the detail of the analysis described herein. In a more rigorous analysis the influence of such factors as infiltration into permeable soils, interception by vegetation, and depression storage should be considered.

Figure 34 shows the depth versus "C_V" curve for the 0-2 and S-3 area. Fewer values were obtained from these areas, however, the curves plotted together in Figure 34 closely parallel each other. The 0-2 and 0-11 areas show a greater percentage of funoff from the storms observed than does the S-3 area. The relative position of the S-3 and 0-2 curves is as expected from the estimated design runoff coefficients, 0.53 and 0.57 respectively. For the 0-11 area, where the estimated design "C_V" factor was 0.53, the curve was very nearly the same as the 0-2 curve. While the 0-11 area was relatively flat and contained some undeveloped areas, open drainage ditches continued to flow into this system for several hours after a storm. This may be ground water or inter-flow from storm water which had infiltrated the surface. Also, the topographic maps from which the "C_V" factors were estimated were 6 years old at the time of the study and the rapid development for industrial use may have increased the runoff coefficient.

From the S-3 and 0-11 watersheds, a limited number of values were obtained for analysis of the peak runoff rate. The "C" versus "I" values are plotted in Figure 35 and separate curves drawn for each area. As in the rational method, two basic assumptions were made: (1) The rate of runoff to the point under consideration is a function of the average rainfall rate during the time required for water to flow from the most remote part of the drainage area to that point, said time of flow being the time of concentration and the duration for the given intensity, and (2) the peak rate of runoff occurs within the time of concentration.

The estimated design runoff coefficient for peak flows, "C", is plotted against the Intensity, "I", for a 10-year frequency storm for Des Moines. The design "I" values were determined for the time of concentration for each area, estimated to be 85 minutes for 0-11 and 40 minutes for S-3. Observed "I" values were for durations selected for each runoff, but never less than those stated above.



RAINFALL INTENSITY
AND
PEAK RUNOFF RELATIONSHIP

FIGURE 35

Because of the limited number of values and scattering of the points, the "C" versus "I" curve is a theoretical curve rather than a curve based entirely on the median of the observed values. The curve used was a straight line passing through the "design" point and through the observed values to the right of the grouping. Values falling above and to the left of this line were assumed to be for storms which did not produce optimum runoff due to the rainfall pattern, antecedent moisture conditions, or other factors. The curve drawn is therefore considered to be conservative for the runoffs observed and may be used as a basis of design for facilities intended to handle lower intensity storms.

Significance of Results

The data obtained shows the runoff coefficient for peak flow, "C", to be dependent upon the intensity of the storm. Although the general relationship of "C" to intensity is shown, it is not adequate to suggest any modification to the present use of the Rational Formula. In fact, the principles of the Rational Formula and accepted average "C" values were used to develop a part of the "C" versus "I" curve shown. The storms monitored during the study period were not of the magnitude which would provide data in the range of values meaningful to normal design practice. A much longer monitoring period would be required, which would include storms exceeding the 10-year frequency as well as providing many additional observations.

The data does shed some light on runoff coefficients which could be expected from lower intensity storms, and is useful in evaluating the amount of runoff which should be intercepted where collection and treatment of the higher intensity storms is not feasible. Such a case may be existing sewer systems in heavily built-up business districts where infrequent overflow of high intensity storms may be more acceptable than providing interceptor facilities for storms of the 10-year frequency range.

If treatment of separate or combined storm runoff is dictated by local conditions, a basis for estimating the total volume of flow is also needed. Based on the findings of this study, the total depthof rainfall provides the best means of estimating the volume of runoff. The accepted runoff coefficients for the Rational Formula are assumed to apply to the 6-inch, 24-hour storm used in this study. Storms of this magnitude are very infrequent, however, and information as to the "C_V" factors applicable to storms which occur much more frequently is needed.

The curve developed for Des Moines 20th Street Storm, Sewer, Station 0-11, shows that if a "C_V" of 0.53 is applicable to a 6-inch accumulation in 24 hours, the average annual 24-hour storm of 2.72 inches would produce a "C_V" of 0.25. This type of information is valuable in determining the requirements for treatment of combined or separate storm runoff, for evaluating the degree of treatment afforded by existing interceptor and treatment facilities, and for evaluating annual runoff quantities and pollutional loads.

RAINFALL INTENSITY DISTRIBUTION

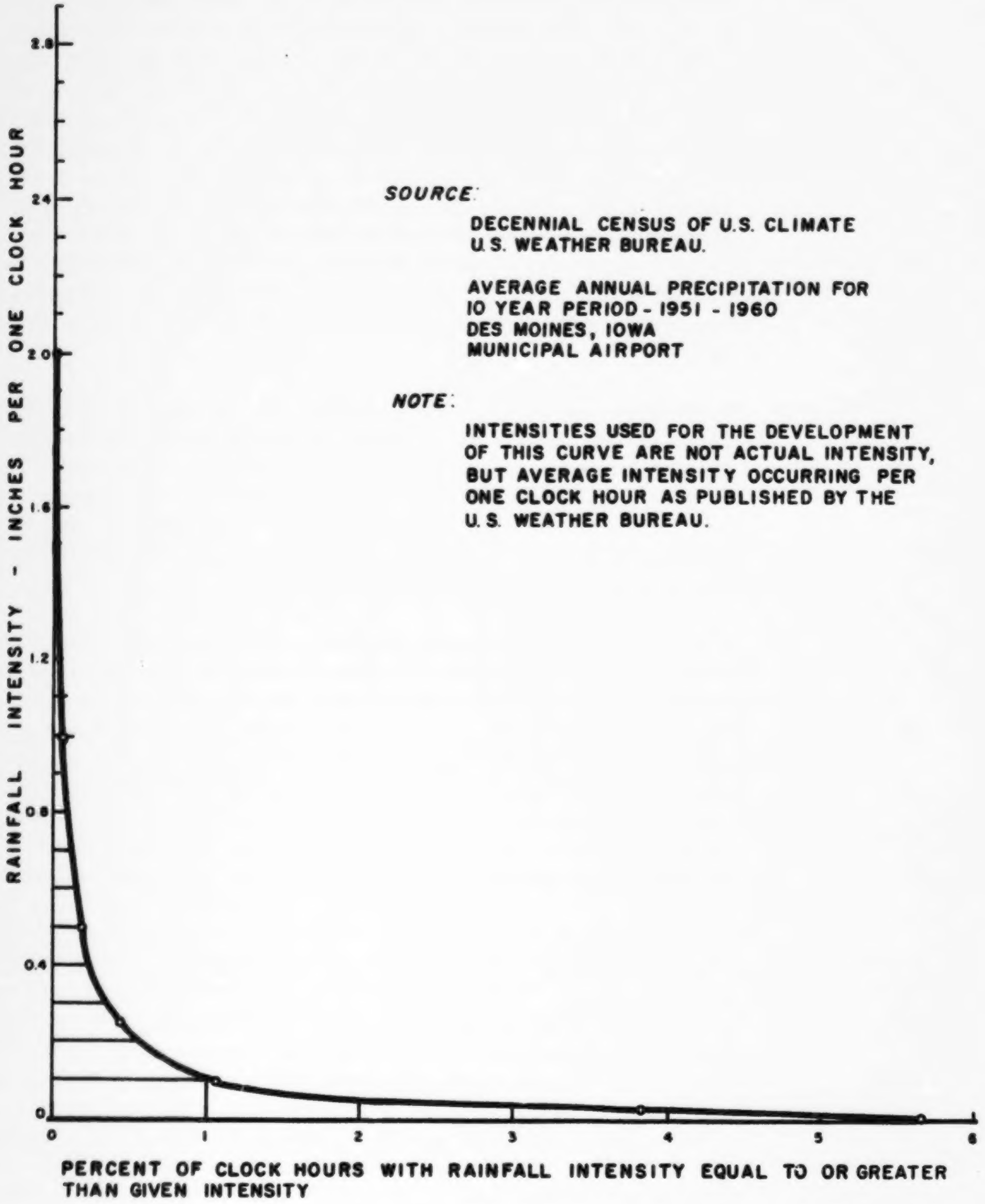
Included in this part of the study was an analysis of the distribution of rainfall intensities with regard to (1) the total depth of rainfall, and (2) the percent of clock hours in which given intensities occur.

Figure 36 shows the relationship of rainfall intensity to the percent of clock hours in which a given intensity is equaled or exceeded. This relationship is similar to that developed for Kansas City, Missouri, by Benjes et. al.(4) Data for the curve shown was developed from published rainfall records for Des Moines for the 10-year period of 1951 to 1960. Rainfall intensities per clock hour were grouped into ranges of intensities as shown in Table 8. The intensity shown is the average intensity occurring over one clock hour as recorded and published by the Weather Bureau. The percentage of the clock hours in which the hourly rainfall exceeded the low value for each range was calculated and plotted as shown in Figure 36.

Table 8
CLOCK-HOUR PRECIPITATION INTENSITY DISTRIBUTION

Intensity Range Inches-Hour	Avg. No. of Clock-Hours Annually	% of Time in Range	Intensity In. / Hour	% of Time Intensity Exceeded
0.01	162	1.85	0.01	5.57
0.02-0.09	241	2.75	0.02	3.72
0.10-0.24	57	0.65	0.10	0.97
0.25-0.49	19	0.21	0.25	0.32
0.50-0.99	8	0.09	0.50	0.11
1.00-1.99	1.7*	0.02	1.00	0.02
Greater than 2.00	0.13*	0	2.00	0

*Calculated on the basis of return frequency of storms for Des Moines, as determined from published and unpublished data from the U. S. Weather Bureau.



SOURCE:

DECENNIAL CENSUS OF U.S. CLIMATE
U.S. WEATHER BUREAU.

AVERAGE ANNUAL PRECIPITATION FOR
10 YEAR PERIOD - 1951 - 1960
DES MOINES, IOWA
MUNICIPAL AIRPORT

NOTE:

INTENSITIES USED FOR THE DEVELOPMENT OF THIS CURVE ARE NOT ACTUAL INTENSITY, BUT AVERAGE INTENSITY OCCURRING PER ONE CLOCK HOUR AS PUBLISHED BY THE U.S. WEATHER BUREAU.

DISTRIBUTION OF RAINFALL INTENSITY
WITH
RESPECT TO TIME

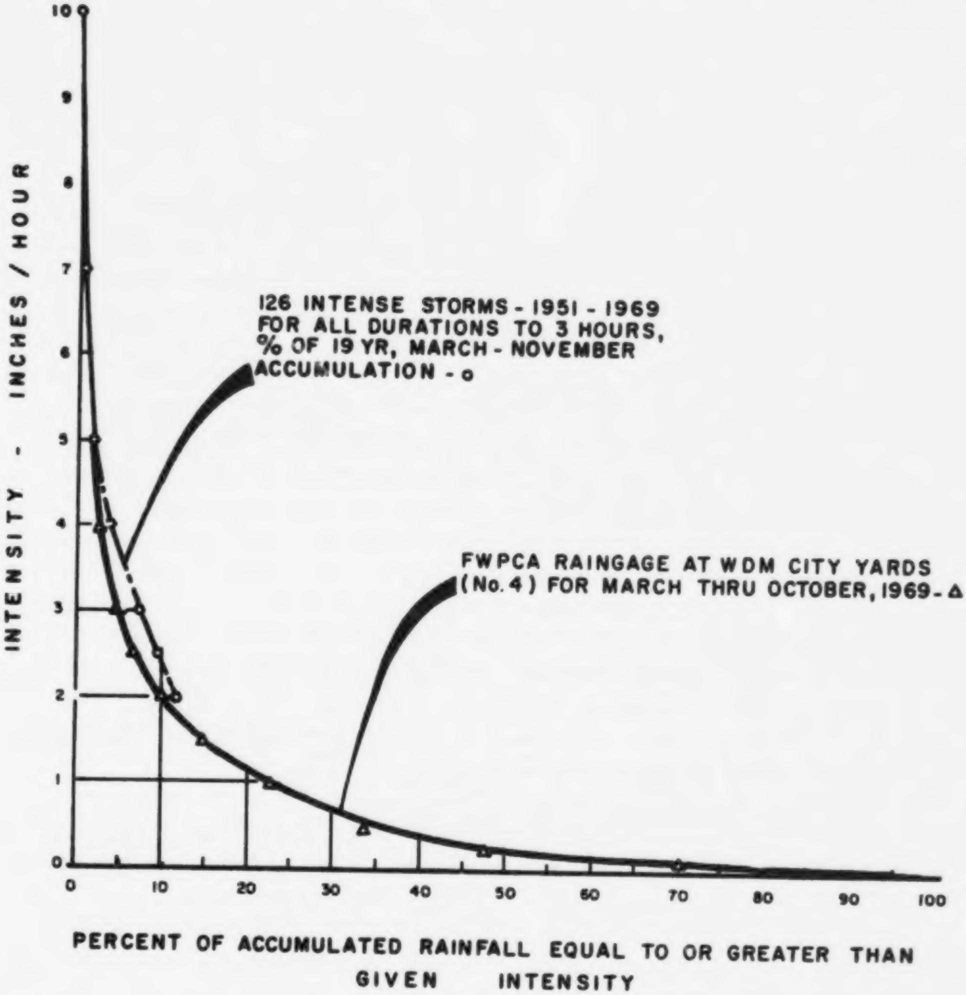
FIGURE 36

The foregoing data is useful in determining the amount of combined sewer overflow which might be expected annually from a given watershed. Given the area of the watershed, the capacity of the combined sewer, and coefficients of runoff as previously described, the intensity of rainfall that can be contained by the system can be calculated by the Rational Formula. From this, the percent of time annually that overflow will occur can be predicted by use of a curve such as shown in Figure 36. For example, if it were determined that a given combined sewer would contain the runoff from a rainfall of up to 0.8 inches per hour intensity before overflowing, overflow would be expected to occur 0.1 percent of the clock hours or approximately 9 hours annually. From this, an estimate of sanitary sewage overflow is possible.

A relationship of the same nature was developed for the total rainfall depth which occurs at or greater than a given intensity. This relationship is shown in Figure 37. Two sources were used to develop this curve. Data from one of the supplemental rain gauges for this study were used to develop the lower intensity values of the curve and inten storms recorded at the Des Moines Airport Weather Bureau Station were used for the high intensity ranges.

The rainfall recorded at the gauge located at the West Des Moines Cit yards was tabulated for the period from March 1 through October 31, 1969. The procedure was similar to that described for the runoff analysis; i. e., the time of day and depth were tabulated for all break points on the chart trace. It was then a relatively simple computer analysis to determine the intensities between the given points and to tabulate the depth of precipitation occurring within specified intensity ranges. The values thus obtained were then plotted as the percentage of total depth which exceeded the lower limit of each intensity range.

The higher intensity section of the curve was developed from 126 intense storms occuring from 1951 through July 1969. The record contained accumulated depth of rainfall covering time intervals from 5 minutes to 3 hours for each storm. These were first ranked in descen ing order of magnitude, listing both the depth and the average intensit The depth of precipitation which fell at or above given intensity values was then accumulated for all of the 126 intense storms. Omitting the winter months during which no intense storms occurred, the 19 years accumulation of March through November rainfall was determined. The accumulated depth occurring at or above each intensity value was then expressed as a percentage of the 19-year accumulated March to November rainfall, and plotted against the intensity values. For intensities above 5 inches per hour, this curve transitioned smoothly into the curve plotted for the lower values.



DISTRIBUTION OF RAINFALL INTENSITY
WITH
RESPECT TO ACCUMULATED RAINFALL

FIGURE 37

The curve shown in Figure 37 may be used in much the same manner as that developed for the clock hours exceeding a given intensity. For example, if a given facility has the capacity to handle the runoff resulting from a rainfall of 1.0 inch per hour intensity, approximately 22 percent of the average March to November rainfall would be expected to occur at intensities greater than this. From this the quantity of overflow can be evaluated.

CONCLUSIONS

The techniques described in this section provide the engineer a means of relating the quantity of combined or separate storm water runoff to rainfall events. Rainfall records are available for all metropolitan areas and are useful for determining local precipitation patterns. Intensity-depth distribution information is helpful, although not generally available. This can be developed, however, and provides a useful tool for evaluation of the effectiveness of alternate levels of treatment as well as for the design of facilities for collecting and treating storm runoff. Runoff coefficients for storms of moderate and low intensity are also desirable. Due to the lack of more appropriate data, studies regarding interception ratios for combined sewer systems must often use assumed or emperical runoff relationships. If storm runoff and combined sewer overflows are to be treated, appropriate coefficients of runoff should be developed for the magnitude of the design storm. Although a treatment facility may be designed hydraulically to contain or pass a 10 or 25-year frequency storm, the storm which will occur once, twice or 12 times per year may be the optimum design for satisfying the requirement of the receiving waters.

The information developed herein has been used in Section X for determining the basis of design for stormwater treatment facilities and for evaluating the effectiveness of the alternate abatement plans.

Because this investigation of rainfall-runoff relationships was only one part of a very comprehensive study relating to combined sewer overflow pollution, procedures were necessary which normally would not be used in a study of this nature. The monitoring program, for instance, was designed to obtain data from many sewer systems within the Des Moines area, and rain gauging stations were not located directly within the watersheds monitored. Also, time and resources limited the amount of effort and detail which could be given to this aspect of the overall study. Because care was taken in reviewing and using the data obtained, the results obtained herein are believed to be accurate and have provided a realistic basis for determining design parameters. The procedures developed herein are sound and may be useful in other areas for evaluating the pollutional effects and the control of combined sewer overflows and urban storm water discharges.

Rainfall-runoff relationships warrant a separate study in which the primary objective is to determine these relationships. Most important is the period of record. Data herein is from one year of monitoring, in which above-normal precipitation occurred. A much longer period of record would be desirable to provide additional data and a broader base for analysis. This would also provide data on higher intensity storms than was obtained during this study. It is much more desirable to have the raingauge stations located within the watersheds being monitored. Runoff monitoring facilities should be more permanent, capable of accurately measuring high variations in discharge, and be easily maintained. Also, the influence of such factors as depression storage, topography, infiltration into pervious soils, and interception by vegetation should be considered. These factors vary widely and have significant effects on the lower intensity rainfalls which contribute a large percentage of ubran storm water discharges due to their frequency.

As a result of the experiences obtained from this study, future rainfall-runoff investigations should be directed toward (1) general studies to determine specific characteristics of specific watersheds or (2) in-depth studies of long duration in which all influences on the lower to medium intensity rainfalls can be determined. The procedures developed herein, however, provide a general approach for determining the magnitude and frequency of urban storm water discharges.

SECTION VIII

RIVER DATA AND ANALYSIS

GENERAL

To establish existing river quality and to obtain data for evaluation of the impact of sewage overflow and storm runoff, a network of river sampling points were established in the project area. The inset in Figure 38 shows the location of the sample stations. It will be noted that the stations bracket the metro area in an attempt to be able to determine and correlate above and below quantitative pollutant values. The station locations were:

Station R-2	Des Moines River near Saylorville
Station R-3	Des Moines River at Euclid Avenue
Station R-4	Des Moines River at Elm Street
Station R-5	Des Moines River at S.E. 14th Street
Station R-6	Des Moines River at Hwy. 46 Bridge
Station R-7	Raccoon River at I-35 Bridge
Station R-8	Raccoon River at 63rd Street
Station R-9	Raccoon River at 5th Street
Station R-10	Walnut Creek at 63rd & Grand
Station R-14	Four Mile Creek at Scott Street
Station R-15	Beaver Creek at Merle Hay Road
Station R-16	Saylor Creek at 12th Street

It would have been ideal to have sampled the river stations frequently and, if possible, concurrently with overflow sampling. Unfortunately project budget limitations allowed for only five samplings. Within the limited funds available, the philosophy was to direct primary effort toward gauging and evaluating overflow and runoff. Additional river quality data was obtained from two other sampling programs in progress at the time of this study. They were:

1. Routine river quality monitoring by the Iowa State Hygienic Laboratory for the State Department of Health.

2. A preimpoundment water quality study for Saylorville Dam. This study was sponsored by the Corps of Engineers and was conducted by the Iowa State University Engineering Research Institute.

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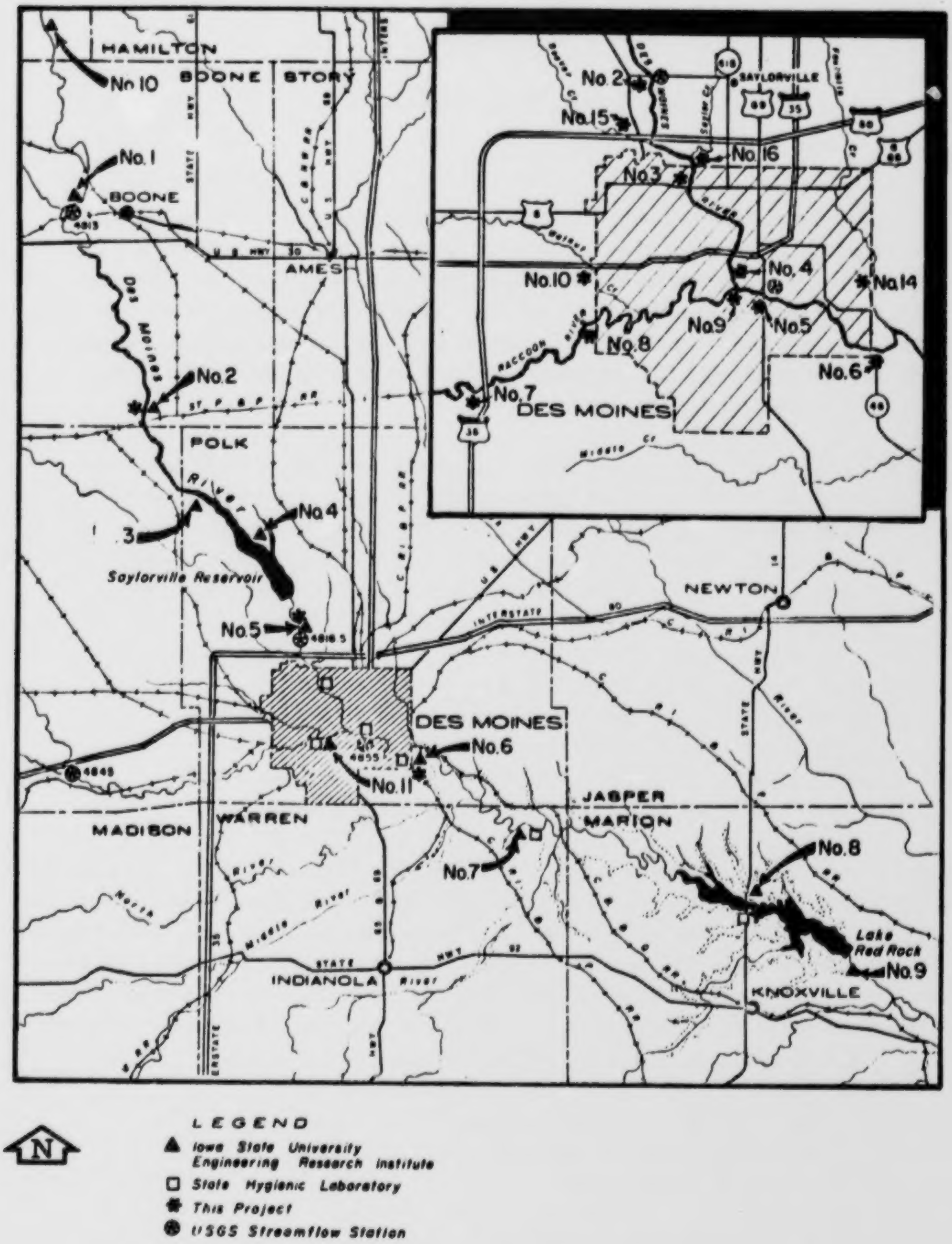
HAMILTON
BOONE
STORY
No.10
No.1
BOONE
4813
AMES
Des Moines River
No.2
POLK
3
No.4
Saylorville Reservoir
No.5
4816.5
DES MOINES
No.6
4855
No.11
4845
NEWTON
JASPER
MARION
MADISON
WARREN
No.7
No.8
Lake Red Rock
No.9
KNOXVILLE
INDIANOLA
No.2
SAYLORVILLE
No.15
No.16
No.3
No.10
No.4
No.14
No.9
No.5
No.8
No.7
No.6
DES MOINES
LEGEND
▲ Iowa State University Engineering Research Institute
☐ State Hygienic Laboratory
✱ This Project
⊛ USGS Streamflow Station
N

LOCATION MAP
RIVER SAMPLING POINTS

FIGURE 38

The State Hygienic Laboratory program began in March 1968. Samples are generally collected on a monthly basis with more frequent sampling during critical periods. Sample stations begin at the upper end of the metro area and extend through the city and into the Red Rock Reservoir area downstream.

The preimpoundment study was begun in July 1967. Samples were collected weekly at 5 stations from Boone to Saylorville. Beginning in April 1969, this sampling schedule was expanded to include intermittent sampling in and below Des Moines. The expanded program was sponsored in part by the research assistance provision of the I. S. U. laboratory contract for this project.

Figure 38 shows sample station locations for the various programs and also locations and identification numbers for U. S. Geological Survey streamflow stations.

Although considerable data is available from the above studies for sample stations outside the project area, the only data presented herein is that for stations within the area. Said data is tabulated in Appendix C. Also presented in Appendix C are figures which show flow, BOD, DO, nitrogen, phosphates and upstream precipitation for Stations R-2, R-5, R-6 and R-9, the stations which generally bracket the metro area. In addition, diurnal DO curves for each station are shown in Appendix C.

SAMPLE SCHEDULING AND PROCEDURE

Five river samplings were accomplished during the project. Four of them, in February, June, August and October, included 28-hour diurnal dissolved oxygen analyses. The April sampling was a high water sampling during spring runoff.

For sampling, the stations were divided into two runs of about 3 to 3-1/2 hours travel time each. Dissolved oxygen, chemical, bacteriological and plankton samples were collected on the first run at mid-day and then DO sampling continued on a 4 hour schedule through 6 additional runs. All samples except those for DO were delivered to the various laboratories immediately after the first run. Dissolved oxygen samples were fixed in the field when collected and then titrated upon return to the project office. During the August and October samplings, midnight samples were collected for sanitary analysis (BOD, solids, nitrogens and phosphates) at selected stations. A comparison of the mid-day and mid-night data is shown in Table 9.

TABLE 9
COMPARISON: DAY - NIGHT RIVER SAMPLE DATA

STATION	D.O.		B.O.D.		SUSPENDED SOLIDS						NITROGEN						PHOSPHATE			
					TOTAL		VOLATILE		FIXED		NH_3		NO_2		NO_3		T. PO_4		O. PO_4	
	DAY	NIGHT	DAY	NIGHT	DAY	NIGHT	DAY	NIGHT	DAY	NIGHT	DAY	NIGHT	DAY	NIGHT	DAY	NIGHT	DAY	NIGHT	DAY	NIGHT
AUGUST 20, 1969																				
R-2 DES MOINES @ SAYLORVILLE	6.90	6.90	11.9	8.8	102	148	18	36	84	112	0.29	0.92	0.01	0.01	0.63	3.22	0.29	0.31	0.18	0.01
R-3 DES MOINES @ EUCLID	6.55	6.80	6.3	8.4							0.36		0.01		0.72		0.26		0.12	
R-4 DES MOINES @ ELM	6.70	7.30	5.9	8.9	87	127	25	30	62	97	0.90	1.50	0.01	0.06	0.68	1.60	1.04	0.27	1.06	0.04
R-5 DES MOINES @ SE 14	7.20	7.35	4.4	7.4	130	168	26	42	104	126	0.24	0.92	0.02	0.02	1.68	1.84	0.75	0.35	0.47	0.34
R-6 DES MOINES @ HWY 46	7.15	6.65	11.4	8.9	130	212	17	35	113	177	0.12	1.33	0.01	0.04	2.14	2.04	0.92	0.60	0.75	0.14
R-7 RACCOON@ I-35	7.80	7.40	7.2	5.6	117	178	14	47	103	131	0.20	0.29	0.02	0.02	3.96	4.57	0.42	0.26	0.07	0.03
R-8 RACCOON@ 63RD ST.	8.25	7.45	8.1	5.4	125	195	15	28	110	167	0.10	0.51	0.03	0.02	3.72	2.57	0.42	0.27	0.25	0.05
R-9 RACCOON@ 5TH ST.	7.40	7.55	3.4	5.0	98	187	8	24	90	163	1.35	0.48	0.01	0.02	4.13	4.14	0.63	0.32	0.53	0.07
R-10 WALNUT CREEK	7.50	7.35	8.7	3.5	254	183	17	40	237	143	0.29	0.46	0.06	0.05	0.86	2.05	0.59	0.29	0.39	0.10
R-14 FOURMILE CREEK	8.25	7.45	3.9	4.1	67	170	19	21	48	149	1.19	0.65	0.02	0.07	1.08	1.50	1.49	1.65	2.18	1.60
R-15 BEAVER CREEK	7.70	7.65	4.5	2.7	93	153	20	24	73	129	0.17	0.70	0.04	0.03	3.65	4.94	0.36	0.29	0.23	0.10
R-16 SAYLOR CREEK	3.20	4.60	19.9	6.1	175	48	25	15	150	33	0.93	0.48	0.27	0.14	1.24	1.94	1.34	0.86	1.59	0.85
OCTOBER 16, 1969																				
R-2 DES MOINES @ SAYLORVILLE	12.00	12.55	19.3	16.2	57	77	22	32	35	45	0.36	0.48	T	T	0.05	0.03	0.26	0.42	0.04	0.10
R-3 DES MOINES @ EUCLID	10.70	13.15	13.6														0.29		0.06	
R-4 DES MOINES @ ELM	11.50	11.50	12.6		43		21		22		0.33		T		0.01		0.68		0.11	
R-5 DES MOINES @ SE 14	12.15	11.20	13.8	11.8	57	51	21	28	36	23	0.56	0.39	T	T	0.10	0.19	0.48	0.33	0.11	0.08
R-6 DES MOINES @ HWY 46	11.30	10.35	12.7	9.1	36	49	20	26	16	23	0.75	0.93	T	T	0.36	0.52	0.38	0.98	0.54	1.29
R-7 RACCOON @ I-35	11.70	11.35	9.0		35		32		3		0.29		T		0.36		0.51		0.11	
R-8 RACCOON @ 63RD ST.	12.25	11.20	7.8								0.47		0		0.57		0.42		0.08	
R-9 RACCOON @ 5TH ST.	11.00	11.50	7.3		38		21		17		0.41		0		0.44		0.43		0.09	
R-10 WALNUT CREEK	10.50	10.65	3.6		40		18		22		0.28		0		0.43		0.91		0.83	
R-14 FOURMILE CREEK	11.25	10.50	4.7		9		7		2		0.82		0.02		3.18		6.44		5.55	
R-15 BEAVER CREEK	11.65	10.80	5.1		22		7		15		0.30		0		2.78		0.64		0.64	
R-16 SAYLOR CREEK	5.10	5.75	5.7	3.6	83	19	17	11	66	8	0.45	0.25	0.04	0.04	0.35	0.49	1.68	1.23	0.49	1.77

T = TRACE

DISCUSSION OF RIVER DATA

Reference to the figures and diurnal DO curves in Appendix C shows that the Des Moines River, above the metro area, normally carries a high BOD load. In fact, the river survives without apparent septicity for extensive periods during which BOD concentrations exceed DO levels. Average values for the periods of available record were as follows:

		R-2 Des Moines at Saylorville	R-9 Raccoon at 5th Street
Available Record		June 1967 to October 1969	March 1968 to October 1969
BOD	mg/L	10.33	6.73
DO	mg/L	9.95	9.20
NO_3	mg/L	2.83	3.88
$O.PO_4$	mg/L	0.44	0.46

Fortunately, the available record covers both dry and wet water years so a comparison of data was obtained. During the low water year of 1967-68, record minimum discharge was recorded at Station R-2. The Raccoon River was also very low, though not at record minimum. Representative average concentrations for the wet and dry periods have been used to estimate annual BOD, nitrate and phosphate loads incoming to the Des Moines metropolitan area. These estimates are given in Table 10.

Comparative BOD, nitrate and ortho-phosphate loads for the control station below the metropolitan area, Station R-6, are shown in Table 11.

The respective drainage areas for the two rivers and unit annual runoff values are shown in the following summary. The unit values are comparable to similar published data.

TABLE 10

ESTIMATED ANNUAL RIVER LOADINGS ABOVE DES MOINES METROPOLITAN AREA

	Annual Runoff (Ac. Ft.)	Annual BOD (Lbs.)	Annual Nitrate (Lbs.)	Annual Ortho-Phosphate (Lbs.)
Low Water Year				
Des Moines River	338,200	11,900,000	460,000	368,000
Raccoon River	206,800	3,649,000	1,971,000	225,000
Total	545,000	15,549,000	2,431,000	593,000
High Water Year (1969)				
Des Moines River	3,739,600	63,900,000	40,500,000	5,060,000
Raccoon River	2,050,000	36,170,000	19,532,000	2,232,000
Total	5,789,600	100,070,000	60,032,000	7,292,000
Average				
Des Moines River (1961-69)	1,573,000	50,000,000	14,000,000	2,000,000
Raccoon River (1915-69)	884,000	15,225,000	8,222,000	940,000
Total	2,457,000	65,225,000	22,222,000	2,940,000

TABLE 11

ESTIMATED ANNUAL RIVER LOADINGS BELOW DES MOINES METROPOLITAN AREA

	Annual Runoff (Ac. Ft)	Annual BOD (Lbs.)	Annual Nitrate (Lbs.)	Annual Ortho-Phosphate (Lbs.)
High Water Year-1969	5,847,000	147,000,000	53,200,000	16,600,000
Average (1940-68)	2,785,000	70,000,000	25,400,000	7,950,000

	Des Moines River	Raccoon River
Drainage Area	3,738,000 Acres	2,202,000 Acres
Unit Average Runoff	0.42 Ac.Ft/Acre	0.40 Ac.Ft/Acre
Unit BOD	13.40 Lbs/Acre	6.93 Lbs/Acre
Unit NO_3	3.75 Lbs/Acre	3.74 Lbs/Acre
Unit O. PO_4	0.54 Lbs/Acre	0.42 Lbs/Acre

Additional river data is shown in Table 12, Comparative Data From Project River Sampling." These data illustrate the problem often encountered in attempting to equate and/or summate consecutive river station data. For example, the sum of the BOD data for the two upstream stations, R-2 and R-7, was on two occasions greater than the BOD at the downstream station R-6. The same is true for nitrates, phosphates and plankton. Extensive speculation is not offered as explanation for the apparent discrepancies. Some observations are made, however, as follows:

> The "sometimes" decrease in organic load through the metro area may be attributable to treatment realized in the low head impoundments at Scott and Center Streets on the Des Moines River and just below Fleur Drive on the Raccoon. To some extent these impoundments may be serving as intermittent sedimentation and stabilization units.
>
> All BOD data, including that used from the two other studies, was obtained from unfiltered samples. However, since the analytical technique was the same for all samples, the relative magnitude of the data should not be affected.
>
> There has been some speculation that treated wastewater effluents may exert an antagonistic or retardant effect on the BOD exertion rate of the receiving stream. If true, this may be due to surfactants or to the expected lower exertion rate of the effluent. In this regard,the decreased BOD in 4 or 5 measurements between R-5 and R-6 is of interest. Increased loads between the summation of R-4 plus R-9 versus R-5 are likely due to raw and combined sewage bypassing the intervening area.
>
> Another, and probably the most practical, possibility for the discrepancies is the fact that the data is biological and biochemical in nature and such data does not always provide predictable comparative summations.

TABLE 12
COMPARATIVE DATA FROM PROJECT RIVER SAMPLING

	RACCOON RIVER			DES MOINES RIVER					Σ
	I-35	63RD ST.	5TH ST.	SAYLORVILLE	EUCLID	ELM ST.	SE 14TH ST.	HWY. 46	
DATE	R-7	R-8	R-9	R-2	R-3	R-4	R-5	R-6	R-7 & R-2
B.O.D.-LBS/DAY									
2/6/69	27,731	30,374	25,185	8,247	18,555	22,687	105,622	61,965	35,978
4/23/69	96,448	103,787	134,189	465,264	483,100	438,423	545,468	466,019	561,748
6/5/69	53,684	65,327	38,161	44,969	64,955	74,948	164,384	119,119	98,653
8/20/69	35,122	39,512	16,585	101,343	53,562	50,246	66,800	173,280	136,465
10/16/69	20,034	17,363	16,251	61,798	43,540	40,339	80,333	73,929	81,832
NO$_3$-LBS/DAY									
2/6/69	16,337	17,563	12,772	17,016	13,552	13,387	27,174	30,348	33,353
4/23/69	228,540	192,687	228,540	467,949	535,949	557,423	891,722	811,087	729,842
6/5/69	52,326	4,851	42,883	260,319	15,939	85,108	17,240	168,196	312,645
8/20/69	19,317	18,146	20,146	5,365	6,132	5,791	25,536	32,528	24,682
10/16/69	1,402	1,269	979	160	224	32	582	2,096	1,562
O. PO$_4$-LBS/DAY									
2/6/69	181	2,037	174	3,134	3,519	324	605	1,280	3,315
4/23/69	12,287	7,737	8,680	34,268	33,284	16,016	23,360	42,570	46,555
6/5/69	815	54	3,098	3,648	3,647	6,595	13,389	15,200	4,463
8/20/69	341	1,220	2,583	1,533	1,022	9,027	7,144	11,400	1,874
10/16/69	245	178	200	128	192	352	640	3,143	373
PLANKTON-ALGAL UNITS PER ML.									
2/6/69	3,440	2,487	4,013	3,531	4,465	3,392	2,316	3,448	
6/5/69	22,751		55,868	33,799		30,837		53,836	
8/20/69	31,425		27,017		58,554	50,670	51,037	48,155	
10/16/69	51,970		76,330	147,150		118,267	102,499	63,330	

DES MOINES METRO AREA LOAD

Estimated daily and annual metro area loads have been computed from the runoff and sampling data presented in Sections VI, VII and IX. Development of the numbers was based on the following criteria:

1. The wastewater treatment plant effluent discharge during dry weather conditions is based upon average flows and effluent quality during the study period.

2. Under "wet" dry weather conditions, the wastewater treatment plant effluent loads is based on the hydraulic capacity of the plant and the average plant effluent BOD during the "wet" dry weather period in May, June, and July 1969.

3. The volume of combined sewer overflow is based on the area presently served by combined sewers. Runoff coefficients, average annual rainfall patterns, and combined sewer overflow quality as determined by this study served as a basis for these computations.

4. Urban storm water discharges were also computed from data developed in this study. The depth and frequency of rainfall events were average annual values obtained from U. S. Weather Bureau records.

The estimated daily metro area loads for the considered conditions are tabulated in Table 13. Projecting the data in Table 13, the estimated average annual loads from the Des Moines area are 11,385,000 lbs. of BOD, 703,640 lbs. of nitrate and 3,092,600 lbs. of ortho phosphate. These summations and the conditions on which they are based are shown in Table 14.

It is recognized that these figures don't match the differences in the summations of the data in the Tables 10 and 11. Possible explanation of the BOD discrepancy have already been discussed. Another thought is to apply unit concentrations to the unaccounted for flow increase between the summation of R-2 plus R-7 and R-6. The increase is probably from the intervening creeks. Using the nitrate and phosphate data for the largest of the creeks, Beaver Creek (R-15), an additional load of 2,860,000 lbs/year of nitrate and 390,000 lbs/year of phosphate are obtained. This brings the nitrate value close in line with the annual river averages. The phosphate data is still unbalanced, however. The explanation for this is unknown.

TABLE 13
PRESENT DAILY METRO AREA DISCHARGES
WEATHER CONDITION

SOURCE	DRY WEATHER	"WET" DRY WEATHER	6" RAIN	2.72" RAIN	1.50" RAIN	0.75" RAIN	0.375" RAIN	0.175" RAIN*
WWTP EFFLUENT								
FLOW, Ac-Ft	108**	154**		VALUES ASSUMED CONSTANT DURING ALL WET WEATHER CONDITIONS				
BOD, LBS.	15,800	20,800						
NO_3, LBS.	1,560	2,200						
$O.PO_4$, LBS.	6,760	9,600						
"WET" DRY WEATHER OVERFLOW								
FLOW, Ac-Ft	NONE	66		VALUES ASSUMED CONSTANT DURING ALL WET WEATHER CONDITIONS				
BOD, LBS.		20,700						
NO_3, LBS.		90						
$O.PO_4$, LBS.		2,440						
WET WEATHER COMBINED OVERFLOW								
FLOW, Ac-Ft	NONE	NONE	1,011	299	100	10	NONE**	NONE **
BOD, LBS.			82,100	40,500	20,300	2,710		
NO_3, LBS.				243	136	18		
$O.PO_4$, LBS.				6,350	2,440	271		
STORM RUNOFF, OVERLAND & STORM SEWER								
FLOW, Ac-Ft	NONE	NONE	12,846	3,580	1,130	495	169	46
BOD, LBS.			523,000	292,000	153,000	80,500	27,500	749
NO_3, LBS.				6,800	3,060	1,610	550	150
$O.PO_4$, LBS.				3,900	1,840	1,010	344	94
TOTALS PER WEATHER CONDITION								
FLOW, Ac-Ft	108	220	14,077	4,099	1,450	725	389	264
BOD, LBS.	15,800	41,500	646,600	374,000	214,800	124,710	69,000	42,249
NO_3, LBS.	1,560	2,290		9,333	5,486	3,918	2,840	2,440
$O.PO_4$, LBS.	6,760	12,040		22,290	16,320	13,321	12,384	12,134

* BELOW THIS INTENSITY, WHICH REPRESENTS THE 0.10" TO 0.25" RANGE, THE AMOUNT OF RUNOFF IS NEGLIGIBLE

** FLOWS TREATED AT WWTP

TABLE 14
SUMMARY OF PRESENT ANNUAL METRO AREA DISCHARGES

CONDITION	DAYS	BOD(LBS.)	NO_3 (LBS)	$O.PO_4$(LBS.)
WWTP EFFLUENT				
DRY WEATHER	257	4,060,600	400,900	1,737,300
"WET" DRY WEATHER	108	2,246,400	237,600	1,036,800
SUBTOTAL	365	6,307,000	638,500	2,774,100
"WET" DRY WEATHER OVERFLOW	108	2,235,600	9,700	263,500
WET WEATHER COMBINED SEWER OVERFLOWS				
2.72" RAIN	1	40,500	240	6,350
1.50" RAIN	5	101,500	680	12,200
0.75" RAIN	12	32,500	220	3,250
0.375" RAIN	18	0	0	0
0.175" RAIN	20	0	0	0
SUBTOTAL	56	174,500	1,140	21,800
URBAN STORM WATER DISCHARGES				
2.72" RAIN	1	292,000	6,800	3,900
1.50" RAIN	5	765,000	15,300	9,200
0.75" RAIN	12	966,000	19,300	12,000
0.375" RAIN	18	495,200	9,900	6,200
0.175" RAIN	20	149,800	3,000	1,900
SUBTOTAL	56	2,668,000	54,300	33,200
TOTAL ANNUAL DISCHARGE	365	11,385,100	703,640	3,092,600

PESTICIDE SURVEY

Pesticide samples were collected during each of the river sampling periods and forwarded to the University of Iowa State Hygienic Laboratory for analysis. The results of these analyses are given in Table 15.

Inspection of Table 15 shows pesticide levels to be relatively low during the June and October survey, indicating that there probably had been little or no application of pesticides for a few weeks preceding the sampling. No pesticides were detected in any of the February samples, the lower limit of detection for chlorinated hydrocarbon insecticides being 0.01 parts per billion. The August sample showed somewhat high concentrations at nearly all stations, and extremely high values were recorded for Stations R-6, the Des Moines River at Highway 46 Bridge (below the city), and for Station 14, Four Mile Creek at Scott Street. An investigation of pesticide usage in the period before the sampling revealed that extensive spraying for flies and mosquitoes was done at the Iowa State Fairgounds in preparation for the fair. Rainfall during that period had carried the pesticides to Four Mile Creek and subsequently the Des Moines River. This situation confirmed previous State Hygienic Laboratory experience which showed that the level of pesticide concentrations in streams was not constant, but varied considerably and reached its highest levels in small streams adjacent to areas where the pesticides are used.

EVALUATION: IMPACT OF METROPOLITAN AREA LOAD

Review of the average annual loadings from the Des Moines metro area during the study period reveals that approximately 75 percent of the annual quantity of BOD, 92 percent of the nitrate and 98 percent of the ortho-phosphate was discharged to the river during non-runoff periods. It should also be noted that about 20 percent of the discharge during the study period was from combined sewer overflow during non-runoff periods. Non-runoff periods represent about 95 percent of calendar year. Wet weather combined sewer overflow, which is the primary subject of concern, represents about 24 percent of the annual BOD load, 10 percent or less of the nitrate and ortho-phosphate loads, and occurs during about 5 percent of the calendar year.

As previously discussed, the river sampling program conducted with this project was limited to five samplings at quarterly intervals. The data collected was useful for evaluating existing water quality; however, it was not sufficient to determine what impact, if any, the Des Moines metro load exerts on the receiving waters. Additional downstream stations and considerably more sampling would be required to make that determination.

TABLE 15
PESTICIDE CONCENTRATIONS
CONCENTRATION IN PARTS PER BILLION

PESTICIDE	RIVER STATION											
	R-2	R-3	R-4	R-5	R-6	R-7	R-8	R-9	R-10	R-14	R-15	R-16
RIVER SAMPLE OF FEBRUARY 6, 1969.												
NO PESTICIDES DETECTED IN ANY SAMPLE, STATIONS R-2 TO R-9												
RIVER SAMPLE OF JUNE 5, 1969.												
DDE	-	.039	.035	.044	.036	-	-	-	-	-	N.S.	N.S.
DIELDRIN	.004	-	-	-	-	.004	.004	.007	. 006	.007		
RIVER SAMPLE OF AUGUST 20, 1969												
DIELDRIN	.006	.008	.015	.028	.23	.011	.010	.034	.021	.20	.25	.007
DDT	.005	-	.042	.10	.93	.003	.005	.048	.015	1.29	.010	.012
DDE	.009	.008	.19	.89	14.4	.020	.032	.49	.020	17.6	.017	.018
op DDT	-	-	.09	.38	9.81	-	-	.29	-	11.2	-	-
op DDE	-	-	.04	.10	2.82	-	-	.14	-	3.30	-	-
RIVER SAMPLE OF OCTOBER 16, 1969												
DIELDRIN	.003	N.S.	.008	.006	.009	N.S.	N.S.	.010	.008	.011	.013	.006
DDT	.007		.006	.004	.007			.008	.013	.011	.011	.012
DDE	.004		.008	.007	.006			.006	.010	.007	.007	.013
DIELDRIN					.009					.009		
DDT					.013					.014		
DDE					.007					.008		

A conjectural appraisal of the probable impact from the metro area loads can be developed by comparing the annual metro area load data to the above and below river data presented herein. It should also be noted that the metro area loads include both combined sewer overflow and overland runoff. The comparison is shown in the following:

Parameter	Low Water Year	High Water Year	Average Water Year
BOD, (lbs)			
Incoming	15,549,000	100,070,000	65,225,000
Metro Area	11,385,100	11,385,100	11,385,100
NO_3 (lbs)			
Incoming	2,431,000	60,032,000	22,222,000
Metro Area	703,640	703,640	703,640
O. PO_4 (lbs)			
Incoming	593,000	7,292,000	2,940,000
Metro Area	3,092,600	3,092,600	3,092,600

The preceding data indicates that the incoming BOD and NO3 loads to the metro area range from only slightly greater to over 80 times the load added in the metro area. The situation reverses for O. PO4 where the metro area addition exceeds the incoming loads during low and average water years. But again, for high water years, the incoming phosphate load is much greater.

Annual loads, although significant in the general picture, do not reflect the impact from potential "shock" loads during runoff periods. The daily load data in Table 13, compared to the measured river loads in Table 12, provides a better indication of potential shock effect. Disregarding the extremes in Table 12, the summated incoming daily load to the metro area is about 36,000 lbs of BOD, 25,000 lbs of NO3, and 3,500 lbs of O. PO4 . From Table 13, it is obvious that the load from any rainfall will greatly exceed the average incoming values. Sanitary sewage and combined sewer overflow alone exceed the incoming BOD by from 150 to 250 percent depending on the rainfall. When overland flow is also considered the percentages increase to 360 and 1700 percent. Obviously, an impact evidenced by accelerated degradation would be expected in the river downstream from the metro area.

The preceding hypothetical situation is not necessarily a correct comparison of data. It is unlikely, through not impossible, that a major rainfall would occur only in the metro area and not also in the contributing watersheds above and below Des Moines. Therefore, it is likely that the relative magnitude of the load increase would be much less, though still significant.

The magnitude and continuity of the incoming loads to the metro area indicate that the Des Moines and Raccoon Rivers are both rich in organic and inorganic nutrient materials. This is also indicated by the high algal densities observed in both streams. Considering these facts, the impact to be anticipated from additional loads, whatever the source, is questionable and speculative. Obviously, even without any additional load, neither river will be a clean running stream until load reduction is accomplished upstream from the Des Moines area. Because of the current emphasis on nutrients and eutrophication, a discussion of these matters relative to the study area appears warranted.

The daily and annual loadings incoming to the Des Moines metropolitan area appear to always be sufficient to maintain an abundant algal growth. This would be especially true for the two new reservoirs, Red Rock and Saylorville. Mackenthun (8) states that "a continued high rate of nutrient supply does not appear to be necessary for continued algal production. After initial stimulus, the recycling of nutrients within the lake basin is sufficient to promote algal blooms for at least a number of years." Sawyer (9) reports that nuisance algal growths can be expected if the average concentration of inorganic nitrogen (NH_3, NO_2, NO_3) exceeds 0.3 MG/L and the average inorganic phosphorous exceeds 0.015 MG/L. These conditions are almost always present in both the Des Moines and Raccoon Rivers. It seems pertinent therefore to wonder if BOD and nutrient additions from the Des Moines area are not just dessert at an already abundant sumptious banquet.

Kuentzel (10) questions the present day public clamor for nitrogen and phosphorous removal and suggests that carbon dixode (CO_2) may be an easier parameter to control in programs intended to inhibit algal growth. Algae do, of course, require free CO_2 for growth; and, the most abundant source of free CO_2 is aerobic decomposition. Mr. Kuentzel suggests that control of bacteria and/or bacterial growth will reduce available CO_2 and inhibit algal growth. This of course reinforces the case for wastewater treatment and disinfection, especially in areas where upstream waters are of high quality.

Ferguson (11) also questions the propriety of forging ahead with elaborate schemes for removal of nitrogen and phosphorous from domestic wastewaters on the assumption that control of these nutrients will effect control of algal growth. Referring to the work by Sawyer (9) and summating known available sources of these nutrients, Mr. Ferguson notes that "a minimum of about 250 million pounds of phosphorous a year would still enter surface waters from natural sources even if all phosphorous from man-generated sources were excluded." He adds that the resulting concentration would be more than four times the minimum concentration needed to induce excessive algal growth.

If the river system above the Des Moines area does in fact carry sufficient nutrients for optimum plankton growth, and this does appear to be the situation, it will be difficult to justify extensive collection and treatment of overflow and runoff waters, at the present time. Evaluation of the annual loads from the Des Moines area reveals that approximately 92 percent of the nitrate and 98 percent of the orthophosphate is discharged to the river during non-runoff periods. For the specific case of Des Moines, about 20 percent of the annual load during the study period was discharged by overflow during non-runoff periods. This is not believed to be the typical situation for Des Moines.

Without the severe infiltration which caused "wet" dry weather overflow an estimated 108 days during the study period, the percentage of the annual untreated loads discharged to the river would be considerably less.

SECTION IX

COMBINED SEWER SEPARATION

SEPARATION STUDY AREA

The study area for combined sewer separation is shown on Figure 40 located at the end of this section. There are 2793 acres in the City of Des Moines that are now served by combined sewers. Of this total, 1836 acres were selected for detailed analysis for combined sewer separation. This "separation study area" includes all the Downtown Core Commercial area on the west side of the Des Moines River, the industrial and warehouse area bordered on the south and east by the Des Moines and Raccoon Rivers, a large residential area with typical public facilities interspersed, and core fringe containing commercial and multi-family areas. Separation of all combined sewers in Des Moines has been designated Combined Sewer Overflow Abatement Plan "A."

DATA UTILIZED FOR STUDY

The data utilized as a basis for study and analysis is as follows:

1. Existing quarter section sewer plats of the separation study area were furnished by the City of Des Moines. These plats are at a scale of 1" = 100' and show the location of existing storm, sanitary and combined sewers. Invert elevations are generally shown at manholes except on a large portion of the existing separate storm sewers and some of the very old combined sewers. In locations where invert elevations were not shown on the plats nor in other City records and it was absolutely necessary for analysis, the invert elevations were measured by field survey.

2. Existing quarter section topographic maps of the study area are at a scale of 1" = 100' with 2-foot contour interval. These topographic maps were prepared in 1963 and 1964 and did not include the interstate highway through the separation study area.

3. Construction plans for the Interstate Highway through the separation study area were furnished by the Iowa State Highway Commission. These plans were utilized to update the topographic maps and the sewer plats in the study area.

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4. City of Des Moines microfilm records of sewer construction drawings affecting the separation study area were utilized to clarify and supplement the sewer plats.

5. Construction drawings of recently constructed storm and sanitary sewers were furnished by the City of Des Moines.

6. Results of a recent storm sewer inlet survey prepared by the City of Des Moines showed the number of inlets at each intersection and were used to check and supplement the inlets shown on the sewer plats.

DESIGN CRITERIA

Storm Sewers

Rainfall Intensity - Duration - Frequency Curves. The Rainfall Intensity-Duration-Frequency Curves used in this study are those prepared by the U. S. Weather Bureau in Technical Paper No. 25 (1955) for 5 and 10 year return periods in Des Moines, Iowa (12). See Figure 39.

Time of Concentration. The theoretical time of concentration (duration) was obtained from a nomograph representing the Hathaway Formula:

$t^{2.14} = 2\,Ln$, where

t = time of concentration in minutes

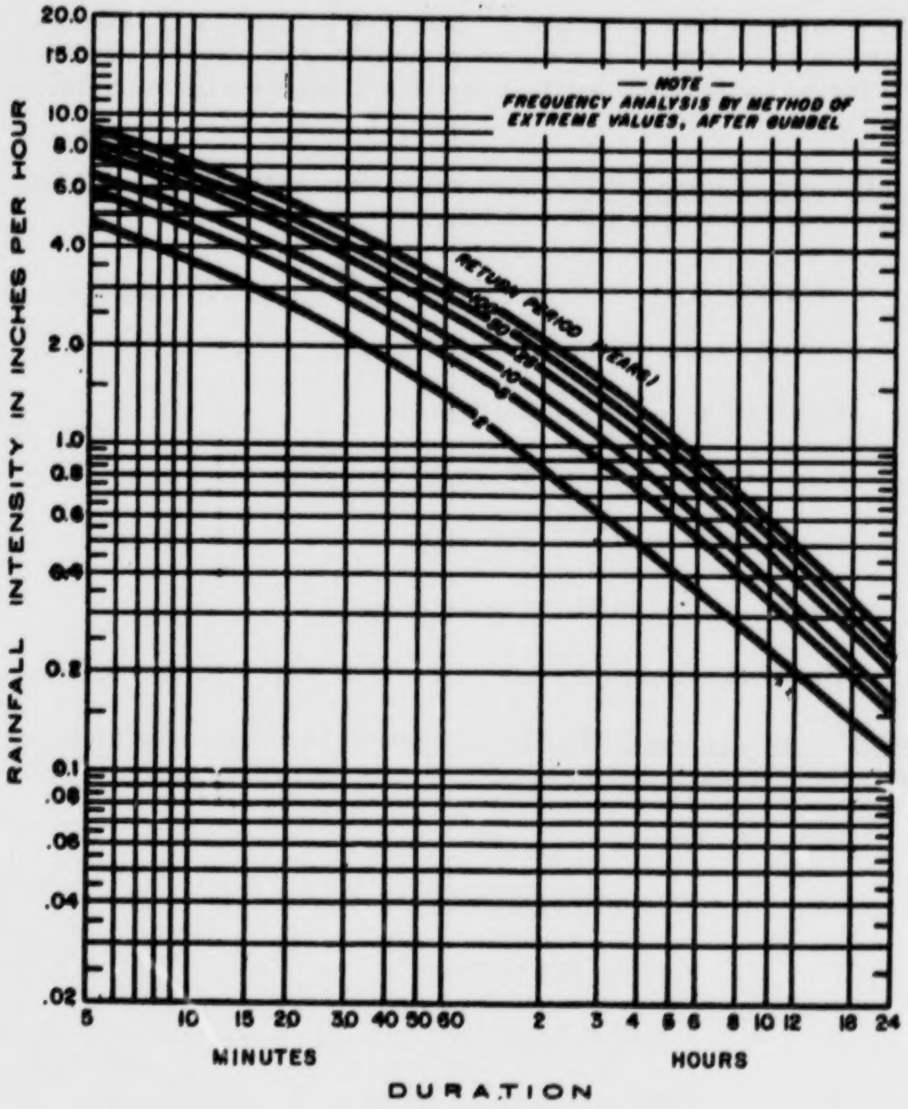
n = retardation coefficient

L = Length in feet

S = Average slope, ft/ft

The retardance coefficient being as follows:

Paved areas	0.01
Bare packed soil	0.10
Sparce grass or moderate rough bare surface	0.30
Average grass cover	0.40
Dense grass cover	0.80



Excerpt from "RAINFALL INTENSITY-DURATION FREQUENCY CURVES", Technical Paper No. 25, U.S. Department of Commerce, Weather Bureau.

RAINFALL INTENSITY-DURATION-FREQUENCY CURVES
DES MOINES, IOWA
1903-1951

FIGURE 39

The minimum time of concentration was limited to 15 minutes. This practice is followed by many municipalities to allow reasonable time for surface retention, surface wetting, lawn storage and gutter storage.

Coefficient of Runoff "C". The following coefficients of runoff "C" were used for storm sewer design.

Surface	"C" Value
Paved areas and building roofs	0.95
Grassed areas level to 1% slope	0.25
Grassed areas 1% to 3% slope	0.35
Grassed areas 3% to 10% slope	0.40
Grassed areas 10% or more slope	0.45
Unpaved alleys, parking areas & drives	0.80

Runoff Computations. Runoff computations were made by using the Rational Method:

$Q = CIA$, where

A = Quantity of flow in cu. ft. /sec.

C = Coefficient of Runoff

I = Rainfall intensity from curves for concentration time (t)

A = Drainage area in acres

Storm Sewer Design Sizing. Storm sewers were sized by using the Manning Formula:

$$Q = A \frac{1.486}{n} R^{2/3} S^{1/2}$$, where

Q = Capacity in cu. ft. /sec.

n = Coefficient of roughness

R = Hydraulic radius of section in ft. $= \frac{\text{area of section}}{\text{wetted perimeter}}$

S = Slope of hydraulic gradient in ft. /ft.

A coefficient of roughness "n" of 0.013 was used for new construction and 0.017 was used for old sewers. Velocities were limited as follows:

Sewers of 54" diameter or less	20 ft. /sec.
Sewers of 60" diameter or larger	22 ft. /sec.

A minimum size of 15" diameter was used in sizing all proposed storm sewers.

Sanitary Sewers

1. The following criteria was used for sanitary sewer design: An assumed population density of 15 persons per acre was used as a basis for sizing proposed sanitary sewer lines.
2. Average daily flow was based on 100 gal. /capita/day.
3. Flow in laterals and sub-main sewers was assumed to be 400 percent of average daily flow for domestic sewage plus 150 gal. /capita/day to account for seepage and basement drains or a total of 550 gal. /capita/ day.
4. Flow in mains, trunk and outfall sewers was assumed to be 250 percent of average daily flow for domestic sewage plus 150 gal. /capita/day to account for seepage and basement drains or a total of 400 gal. /capita/ day.
5. Sanitary sewers were sized by using the Manning Formula. A coefficient of roughness "n" of 0.013 was used for new construction. A minimum velocity for pipes flowing full of 2.0 ft. /sec. was used.
6. A minimum sanitary sewer size of 8" diameter was used.

METHOD OF PROCEDURE

Classification of Existing Sewers

The location of all known existing inlets were plotted on both the topographic maps and the sewer plats. Using the sewer plats, all sewers carrying combined sewage, only sanitary sewage or only storm water were designated on work drawing by color codes. All sewers carrying only storm water and not discharging into a combined sewer were then eliminated from further study. All sewers carrying only sanitary sewage were also eliminated from further study except where the area served contributed to a proposed new sanitary sewer system or linkage.

The boundaries of areas served by combined sewers or storm sewers discharging into combined sewers were then outlined on the topographic maps. The combined sewer systems were analyzed to determine the most efficient method of separation, either (1) new storm sewer systems with existing combined sewers used only for sanitary flows or (2) new storm sewer systems in combination with existing combined trunk sewers of sufficient size to handle design storm flows with the sanitary sewage rerouted into new sanitary systems.

Storm Sewers

Using the topographic maps, the drainage areas contributing to all intersections of points of impact within the existing combined sewer area were outlined. Some 4000 individual drainage areas were involved in the separation study area. Each area outlined on the topographic maps was then measured to determine the acres served.

Storm sewer systems were then laid out at a scale of 1" = 400' for all areas presently served by combined sewers. Existing storm sewers within those areas were included for analysis together with any large combined sewers that were of sufficient size and in locations appropriate for use as storm sewers. The system layout was later revised where the hydraulic analysis indicated that changes were desirable.

Each point of impact, major change of direction, or change of grade was designated by a number code indicating the system number and the point number. This numbering system is used on Figures 41 through 48. The last two digits indicate the point number and the preceding digits indicate the system numbers (2015 indicates system number 20, point number 15).

The values for runoff factors "C" and time of concentration "t" were then determined for each impact point. The factor "C" was determined by detailed analysis of a number of typical areas and the results applied by using visual comparison for all other similar conditions. The resulting factor "C" for each impact point was determined by compositing the contributing sub-areas as shown in the following example:

Point of Concentration	Location	A Acres	C	AC
723	17th and Clark	0.71	0.60	0.43
		2.16	0.55	1.19
		1.07	0.56	0.60
		1.41	0.55	0.78
		5.35	0.56	3.00

Proposed gradients for storm sewers were then determined by setting up tables for each proposed system showing the surface elvations at each point, the distance between points, the proposed inverts, the depth of cut and the invert elevations of existing sewers crossed. Grades were determined working upstream from the elevation of discharge through the system. After analysis, the proposed or existing storm sewer sizes were included in the table and rechecked for conflicts with other sewers and for minimum cover. When existing sewer invert elevations were unavailable, they were assumed; except where the location of large existing sewers was critical the invert elevations were obtained by field measurement.

Hydraulic analysis was performed by electronic computer. The analysis was repeated as necessary to obtain a satisfactory design. In the analysis of existing systems, it was assumed that adequate inlets were in place. Any existing storm sewer forming a branch of a proposed system was not considered for reconstruction even though it was found to be of inadequate size. Existing storm sewers not contributing to combined sewage flows were not analyzed. Although a separate analysis was made for 5 and 10 year rainstorm return periods, only the results of the 10 year rainstorm was used for pipe sizing and cost estimating. The final results of the hydraulic analysis for system number 9 is included as an example.

The storm sewer systems shown in Figures 40 through 48 are those proposed for the purpose of carrying all of the storm water from the areas now served by combined sewers within the separation study area. These systems generally fall within one or more of the following categories.

(1) An independent system with no interconnection with any existing system discharging into a natural ravine or a river.

(2) A branch system, either existing or proposed, discharging into an existing or proposed system.

SAMPLE HYDRAULIC ANALYSIS FOR STORM SEWERS
10 YEAR STORM

```
RELIEF SEWER NO 9 10 YR STORM DES MOINES IA SEW SEP STY VLH 8 01 69


  STAT EXIST EXIST D A FLOW   AREA   C*A  DUR ACCUM. REQD    CAP  SLOPE LTH   FALL  VEL
FROM TO SIZE  CAP AREA INIT    SUM   SUM  MIN   FLOW PIPE   FULL   0/0  FT.    FT.  FPS
        TYPE  CFS  AC   CFS    AC.                CFS SIZE    CFS


  1   2  1 P   0   5.8  13.3   5.8   3.3 18.4   13.3  24    16.5   .53  395   2.09  5.24
  2   3  1 P   0   6.1  14.4  11.8   6.8 19.5   26.3  24    31.0  1.88  480   9.02  9.87
  3   4  1 P   0   4.6   9.8  16.4   9.0 20.2   34.2  36    41.7   .39  370   1.44  5.89
  4   5  1 P   0   0.0   0.0  16.4   9.0 21.2   33.3  36    48.6   .53  175    .93  6.87
  6   7  1 P   0   3.4   7.6   3.4   1.9 18.4    7.6  18     9.7   .85  315   2.68  5.48
  7   5  1 P   0   2.0   4.9   5.4   3.0 19.3   11.7  21    13.9   .77  195   1.50  5.78
  5   8  1 P   0   0.0   0.0  21.8  12.0 21.6   44.0  36    49.5   .55  840   4.62  7.00
  9  10  1 P   0   4.9  12.0   4.9   2.8 16.0   12.0  21    14.3   .81  330   2.67  5.93
 10  11  1 P   0   2.3   5.4   7.2   4.1 17.2   17.0  24    20.0   .78  345   2.69  6.36
 11   8  1 P   0   5.0  13.1  12.2   7.1 18.0   28.5  27    31.0  1.00  185   1.85  7.79
 12   8  1 P   0   6.2  12.1   6.2   3.3 22.2   12.1  24    14.8   .43  185    .80  4.72
  8  13  1 P   0   0.0   0.0  40.3  22.4 23.3   78.6  36   109.2  2.68  540  14.47 15.45
 14  13  1 P   0   4.7  11.8   4.7   2.7 15.0   11.8  24    12.8   .32  190    .61  4.07
 13  15  1 P   0   0.0   0.0  45.0  25.1 23.8   86.9  30    87.7  4.57  310  14.17 17.86
 15  16  1 P   0   0.0   0.0  45.0  25.1 24.1   86.4  24   120.1 28.20  125  35.25 38.24
DRAINAGE AREA TOTALS
AREA=   44.97  CXAP=    0.00  CXAI=   25.09  DUR=   24.09
FLOW=   86.38  CFS/AC=   1.92
```

Note:

(1) Change Pipe Size for Velocity Limit to 30" PTS. 15-16

(3) An existing system downstream of a proposed sewer extension that is to be enlarged due to its inadequate size.

(4) A system that is an extension of an existing system to serve areas conveniently adjacent to that existing system.

Sanitary Sewers

Where existing combined sewers were to be used for storm water, separate sanitary systems were designed to remove and reroute sanitary sewage from such combined sewers. Gravity sewers were used wherever possible and lift stations included only where no alternative was available or practical. Each system was identified by letters, and various points along the system by numbers; i. e., point two on System A was identified as A02.

The areas contributing to the proposed sanitary systems were outlined and measured. Proposed gradients were then determined by the same method as was used for storm sewers. Sanitary flows were analyzed for each line and lift station. Line sizes and lift station capacities were then determined.

OTHER METHODS OF SEPARATION CONSIDERED

Ponding or Retardation of Storm Runoff

Retardation of combined sewer overflows and storm runoff for the purpose of reducing pollutant loads and/or storage for return to the combined sewer system is considered in subsequent plans. Retardation for the purpose of decreasing the size of storm sewers is not considered, however. The separation study area is an older, heavily developed section of the city where land suitable for retardation basins was not available. Also the terrain in this area is rolling and adequate slope is available for good drainage.

Construction of Sanitary Sewers Within Large Existing Sewers

This method was considered, however, hydraulic analysis of the proposed and existing storm sewers showed that no significant reserve capacity was available in these systems. Where it was found necessary to construct new sanitary sewers, existing storm sewer capacity was not sufficient to permit inserting a new sanitary sewer within the existing storm sewer.

COST ESTIMATE

Method of Estimating

A unit price schedule was prepared to include all probable items of cost. Computer data cards were prepared for each run of each system to reflect the quantities of all items of construction. All lump sum items were listed on separate data cards giving the item description and the lump sum cost. Where items occurred that were not listed in the unit price schedule, the cost of a similar item listed was used and the additional cost was entered as a lump sum item. As an example, where box sewer sections or low head pipe was used due to clearance requirements, the additional cost over a comparable item was entered as a lump sum item. A contingency factor of 15 percent was used on all cost estimates to account for conditions not apparent from the basic data available. The quantities and costs were then computed and listed by the computer. An example of the detailed cost estimates is included.

Cost Items Not Included in the Estimate

(1) The cost of plugging existing inlets now connected to combined sewer systems and the connection of these inlets to a proposed storm system is not included.

(2) The cost of constructing additional inlets in the upper reaches of existing storm sewer systems, where the existing inlets may be inadequate to serve the areas. This is not a proper cost item for sewage separation since effective separation is not dependent upon these additional inlets.

(3) The cost of increasing the existing storm sewers to the sizes shown in the hydraulic analysis as required under the criteria used: This item does not affect separation since those systems are now separated at their origin and the existing under-sized sewers will be connected to the proposed separate storm sewer systems or discharge independently to natural ravines.

(4) The cost of ground and utilities surveys, preparation of final designs and construction supervision.

(5) The cost of right-of-way acquisition.

SAMPLE STORM SEWER COST ANALYSIS

SEWAGE SEPARATION STUDY DES MOINES, IA.
COST ESTIMATE STORM SEWER NO. 9 VLH 9-1-69

DESCRIPTION	QUANTITY	UNIT	UNIT COST	AMOUNT
18 STORM SEWER PIPE IN PLACE	315.00	L.F.	5.12	1612.96
21 STORM SEWER PIPE IN PLACE	525.00	L.F.	5.98	3142.12
24 STORM SEWER PIPE IN PLACE	1595.00	L.F.	7.71	12303.83
27 STORM SEWER PIPE IN PLACE	185.00	L.F.	9.31	1722.35
30 STORM SEWER PIPE IN PLACE	435.00	L.F.	10.64	4628.40
36 STORM SEWER PIPE IN PLACE	1925.00	L.F.	15.29	29442.87
EXCAVATION COST OPEN CUT UNSHORED	1499.25	C.Y.	2.50	3748.12
EXCAVATION COST OPEN CUT SHORED	9888.31	C.Y.	3.00	29664.94
PAVEMENT CUT AND REPLACEMENT	2284.02	S.Y.	9.00	20556.21
MANHOLE STANDARD DEPTH 6FT OR LESS	15.00	EA	300.00	4500.00
MANHOLE EXTRA DEPTH OVER 6FT	55.60	V.F.	35.00	1946.00
SINGLE INLETS	22.00	EA	740.00	16280.00
DOUBLE INLETS	11.00	EA	1160.00	12760.00
EXISTING INLETS TO BE CONNECTED	7.00	EA	360.00	2520.00
WATER MAINS RELOCATED VERTICALLY	3.00	EA	100.00	300.00
WATER MAINS CROSSING ABOVE PIPE	8.00	EA	80.00	640.00
CONCRETE SADDLES FOR SANITARY MAINS	3.00	EA	200.00	600.00
SANITARY MAINS CROSSING ABOVE PIPE	2.00	EA	150.00	300.00
GAS MAINS RELOCATED VERTICALLY	2.00	EA	100.00	200.00
GAS MAINS CROSSING ABOVE PIPE	10.00	EA	80.00	800.00
WATER SERIES RELOCATED VERTICALLY	1.00	EA	50.00	50.00
WATER SERIES CROSSING ABOVE PIPE	10.00	EA	40.00	400.00
SANITARY SERIES RELOCATED VERTICALLY	3.00	EA	80.00	240.00
SANITARY SERIES CROSSING ABOVE PIPE	6.00	EA	80.00	480.00
GAS SERIES RELOCATED VERTICALLY	1.00	EA	50.00	50.00
GAS SERIES CROSSING ABOVE PIPE	10.00	EA	40.00	400.00
UTILITYPOLES RELOCATED	3.00	EA	150.00	450.00
GRASSED AREAS CUT AND REPLACED	122.53	S.Y.	1.00	122.53
TREES TO BE REMOVED AND REPLACED	9.00	S.Y.	250.00	2250.00
HEADWALL FOR 30 IN PIPE		LS		550.00
CONTINGENCY .15				22899.05
			TOTAL	175559.38

Intangible Cost Items Not Included

Certain intangible cost items are involved with construction of separated sewer systems in the existing built-up areas, particularly around the central commercial district. Retail business would be seriously disrupted during construction, with attendant loss of revenue. Detouring and maintaining traffic would be a problem and an inconvenience to motorists would result. Also the noise of construction activities would create an annoyance to adjacent office buildings. Existing underground utilities would present conflicts, requiring close coordination with the operating agencies to maintain service.

Cost Summary

The following Tables are a summary of the estimated construction costs required for separation of storm and sanitary flows in the separation study area:

Proposed Storm Sewer Systems

System	Acres	Construction Cost
1 (Areas 1, 2, 3, &4)	145.74	$ 800,686
5	194.78	757,497
6	6.12	13,880
7	148.31	578,153
8	193.02	86,113
9	44.97	175,559
10	10.13	36,338
11	25.72	99,195
12	26.72	57,671
13	14.55	53,448
14	37.94	5,931
15	103.42	256,460
16	2.86	12,703
17	35.82	77,277
18	13.03	16,688
19	185.80	20,077
20 (Existing System with no Additions)	748.99	0
21	297.97	1,742,710
22 (Existing System with no Additions)	38.83	0
23	154.06	406,991
24	105.89	480,112
25	25.51	79,119
26 (Existing System with no Additions)	16.74	0
27	17.46	50,138

System	Acres		Construction Cost
28	134.73		$ 4,910
29	--		116,465
30	23.54		46,476
31	43.73		159,376
32	30.82		82,276
33	115.79		596,874
34	94.00		555,052
35	124.73		278,552
36	17.27		65,543
37	13.53		34,166
		Total	$ 7,746,436

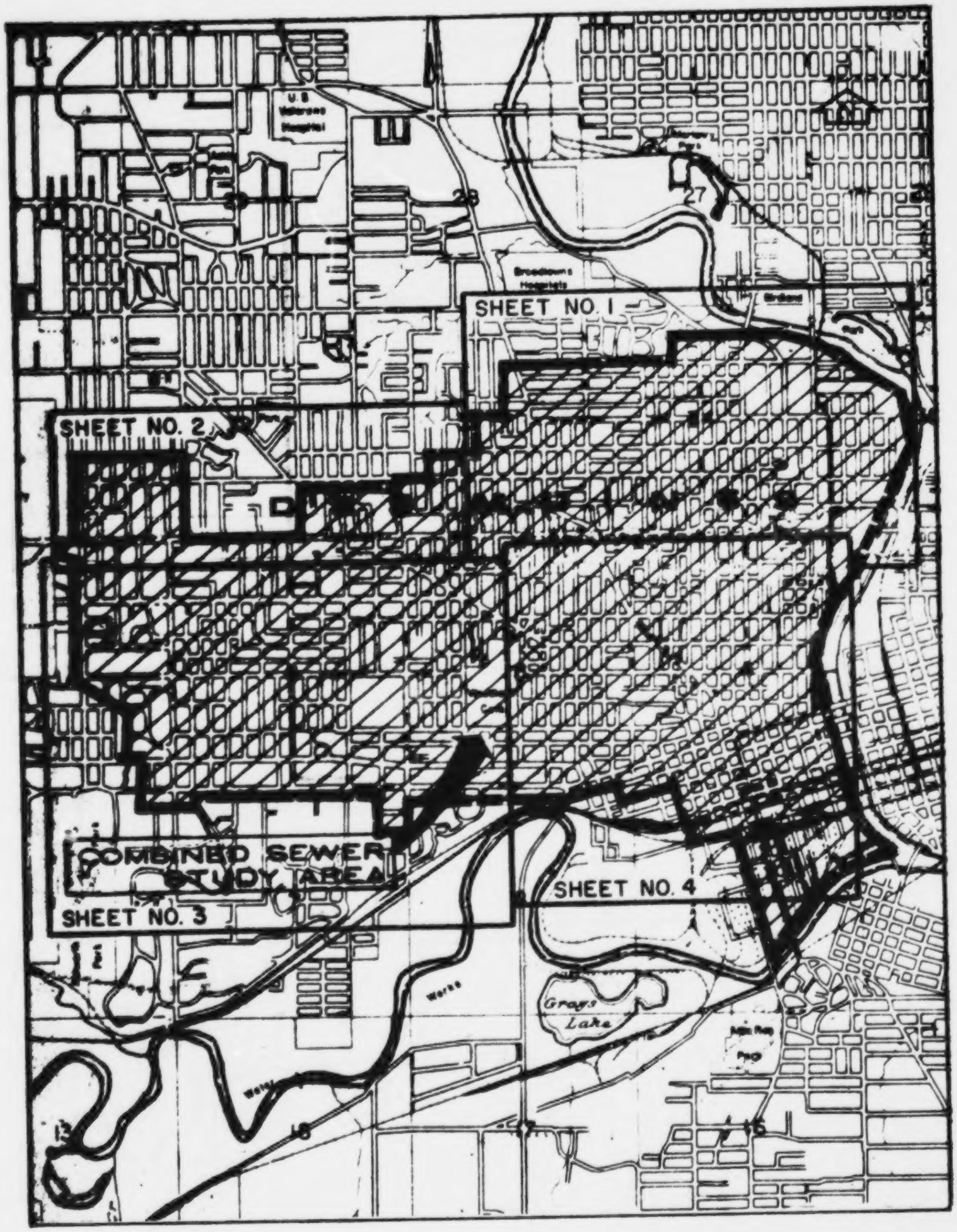
Proposed Sanitary Sewer Systems

System		Construction Cost
A		$ 263,114
B		57,618
C		22,450
D		21,415
E		11,646
F		225,969
G		3,918
	Total	$ 606,130

Total Construction Costs Storm and Sanitary Sewer Systems	$ 8,352,566

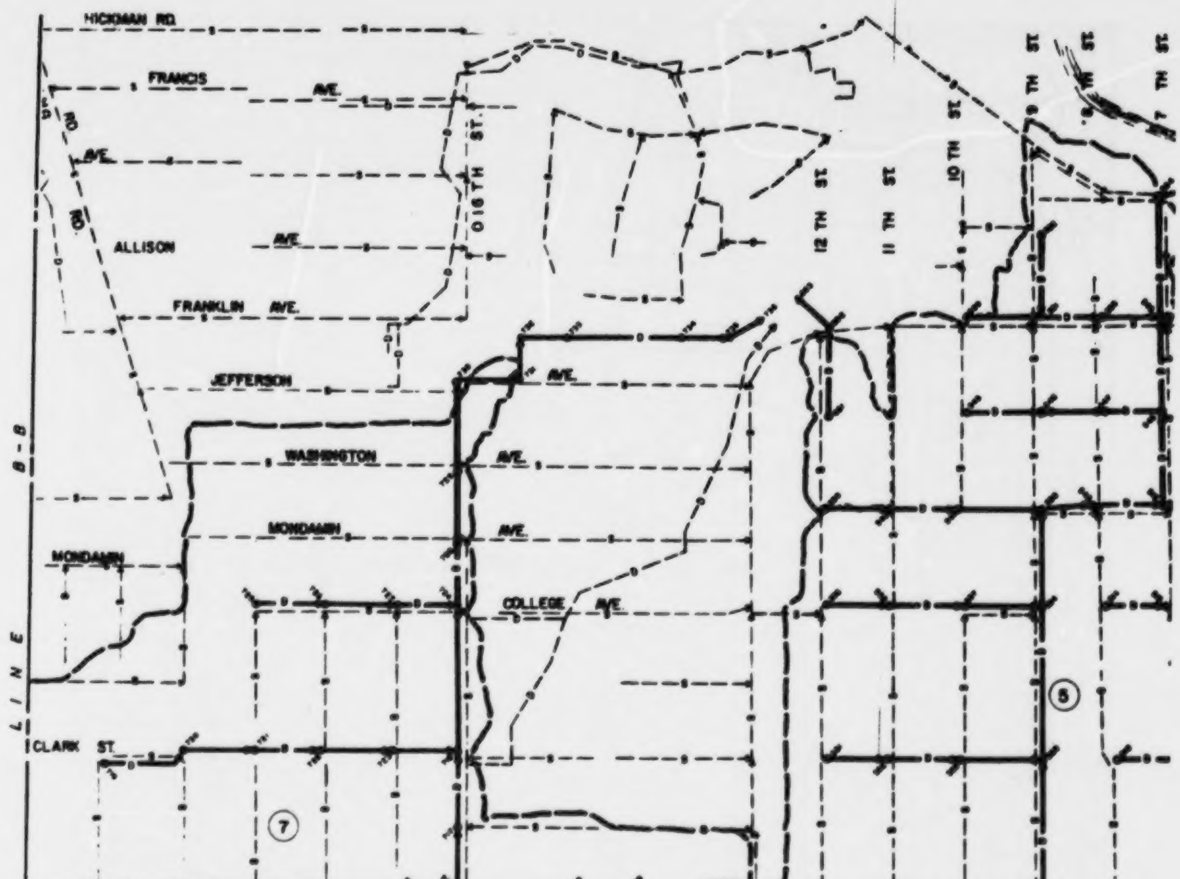
Separation Costs Outside of the Detailed Separation Study Area

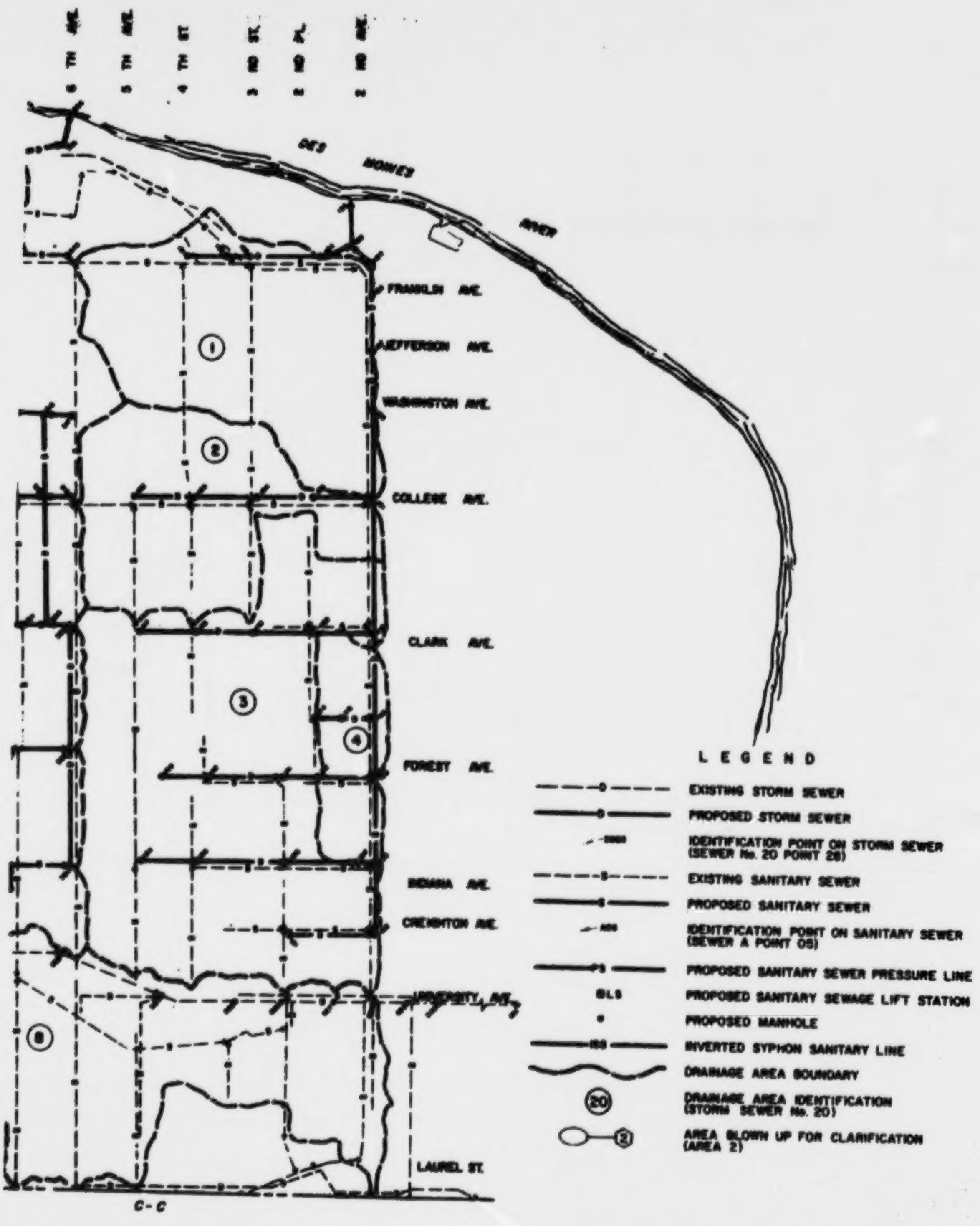
The cost of separating storm runoff from sanitary flows in those areas outside of the detailed separation study area was estimated by measuring all known drainage areas contributing to combined sewer systems and comparing them with similar systems in the separation study area. The actual areas connected were adjusted to account for storm flows picked up and carried by these systems enroute to a discharge point. The areas to be drained were then multiplied by the cost per acre of a similar situation found in one or more systems within the detailed separation study area. It was estimated that an additional 100 acres would be found to be connected to the sanitary systems in addition to those located and measured. An additional 20 percent was added for overall contingencies. The total cost for separation outside of the detailed separation study area was found by this method to be $10,200,000. When added to the cost for the detailed separation study area, the total cost for separation of storm runoff from sanitary flows for the City of Des Moines would be approximately $18,550,000.


U.S. Veterans Hospital
28
27
Broadlawns Hospitals
SHEET NO. 1
SHEET NO. 2
COMBINED SEWER STUDY AREA
SHEET NO. 3
SHEET NO. 4
Grays Lake
13
16
17
15

SEWER SEPARATION PLAN
SHEET INDEX

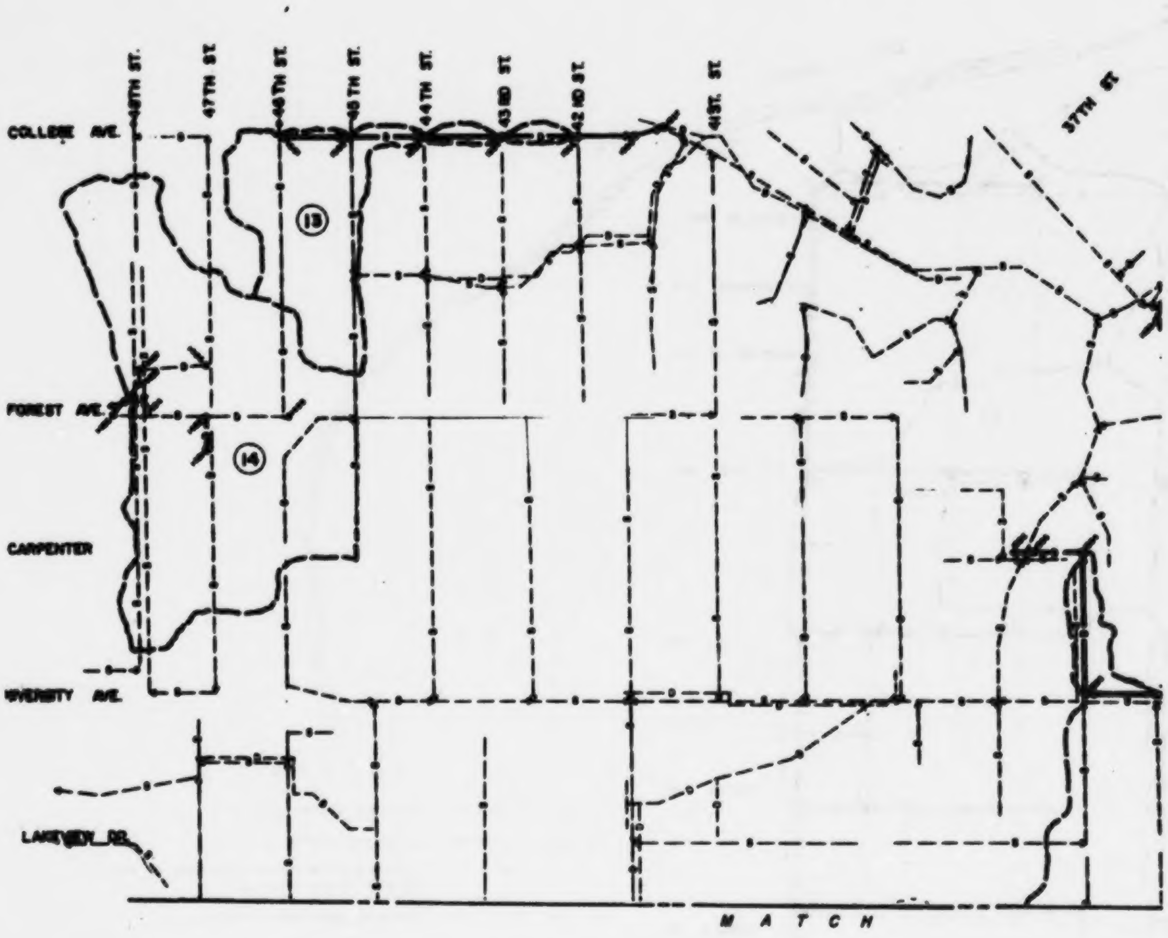
FIGURE 40

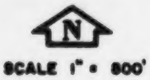

HICKMAN RD.
FRANCIS AVE.
AVE.
ALLISON AVE.
FRANKLIN AVE.
JEFFERSON AVE.
WASHINGTON AVE.
MONDAMIN AVE.
MONDAMIN
COLLEGE AVE.
CLARK ST.
16TH ST.
12 TH ST
11 TH ST
10TH ST
9 TH ST
8 TH ST
7 TH ST
MATCH LINE B-B
MATCH LINE
⑦
⑤
㉙
⑥
㉘
㉙
N
SCALE 1" = 800'

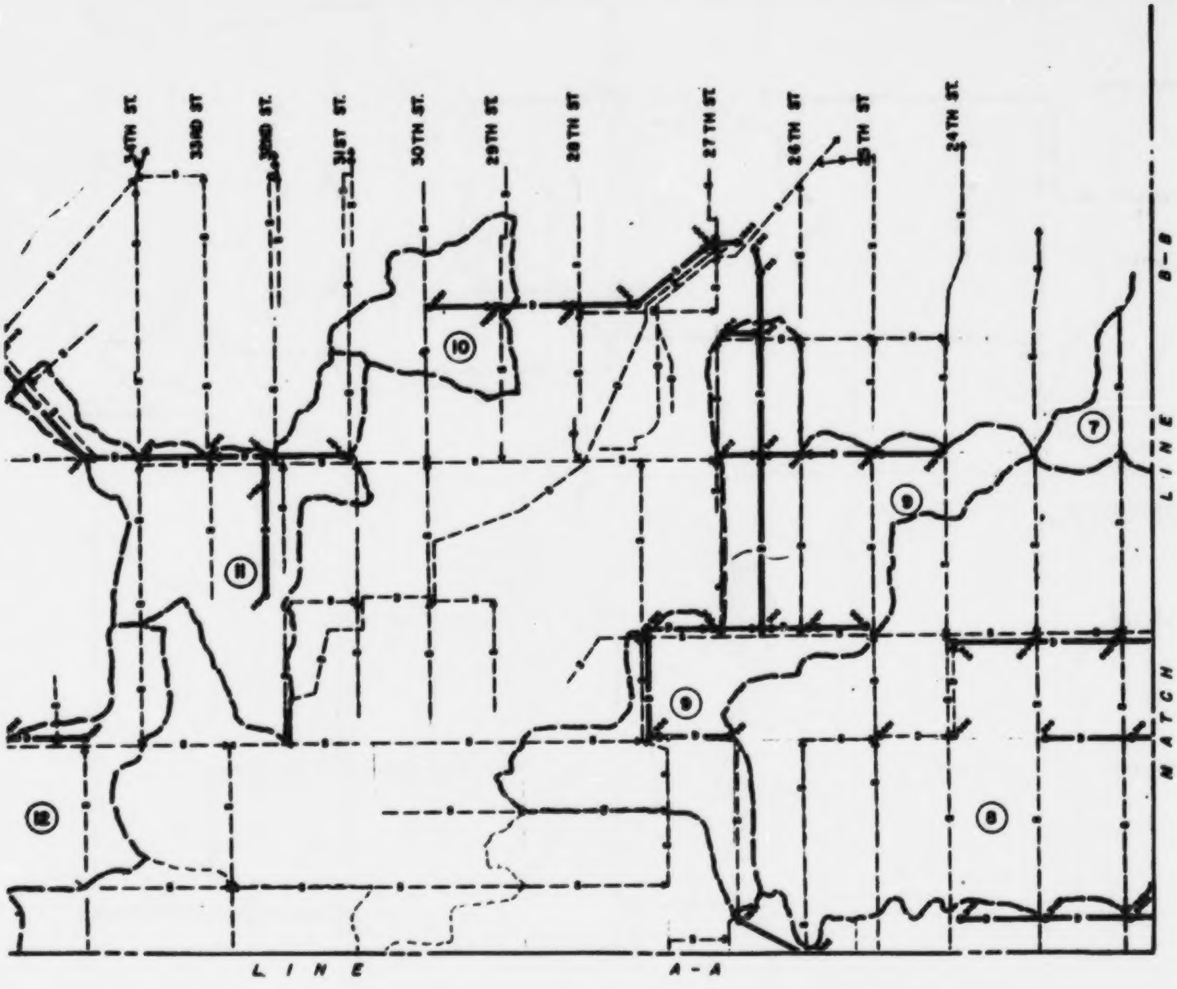

6 TH AVE.
5 TH AVE.
4 TH ST.
3 RD ST.
3 RD PL.
2 ND AVE.
DES MOINES RIVER
FRANKLIN AVE.
JEFFERSON AVE.
WASHINGTON AVE.
COLLEGE AVE.
CLARK AVE.
FOREST AVE.
INDIANA AVE.
CREIGHTON AVE.
UNIVERSITY AVE.
LAUREL ST.
C-C
①
②
③
④
⑧
LEGEND
EXISTING STORM SEWER
PROPOSED STORM SEWER
IDENTIFICATION POINT ON STORM SEWER (SEWER No. 20 POINT 28)
EXISTING SANITARY SEWER
PROPOSED SANITARY SEWER
IDENTIFICATION POINT ON SANITARY SEWER (SEWER A POINT 05)
PROPOSED SANITARY SEWER PRESSURE LINE
SLS
PROPOSED SANITARY SEWAGE LIFT STATION
PROPOSED MANHOLE
INVERTED SYPHON SANITARY LINE
DRAINAGE AREA BOUNDARY
⑳
DRAINAGE AREA IDENTIFICATION (STORM SEWER No. 20)
AREA BLOWN UP FOR CLARIFICATION (AREA 2)

SEWER SEPARATION PLAN
SHEET 1

FIGURE 41

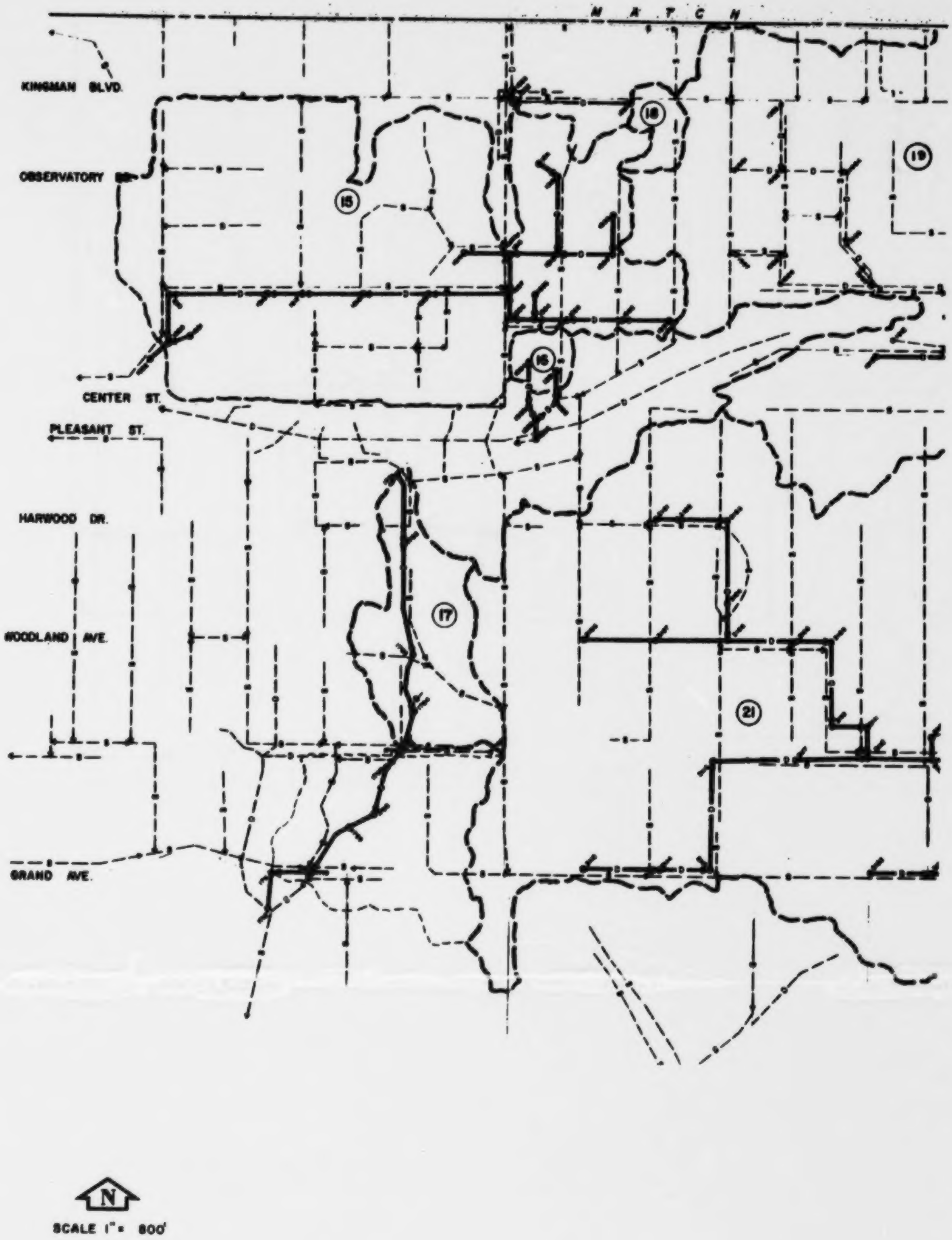

48TH ST.
47TH ST.
46TH ST.
45TH ST.
44TH ST.
43RD ST.
42ND ST.
41ST. ST.
37TH ST.
COLLEGE AVE.
FOREST AVE.
CARPENTER
UNIVERSITY AVE.
LAKEVIEW DR.
13
14
MATCH

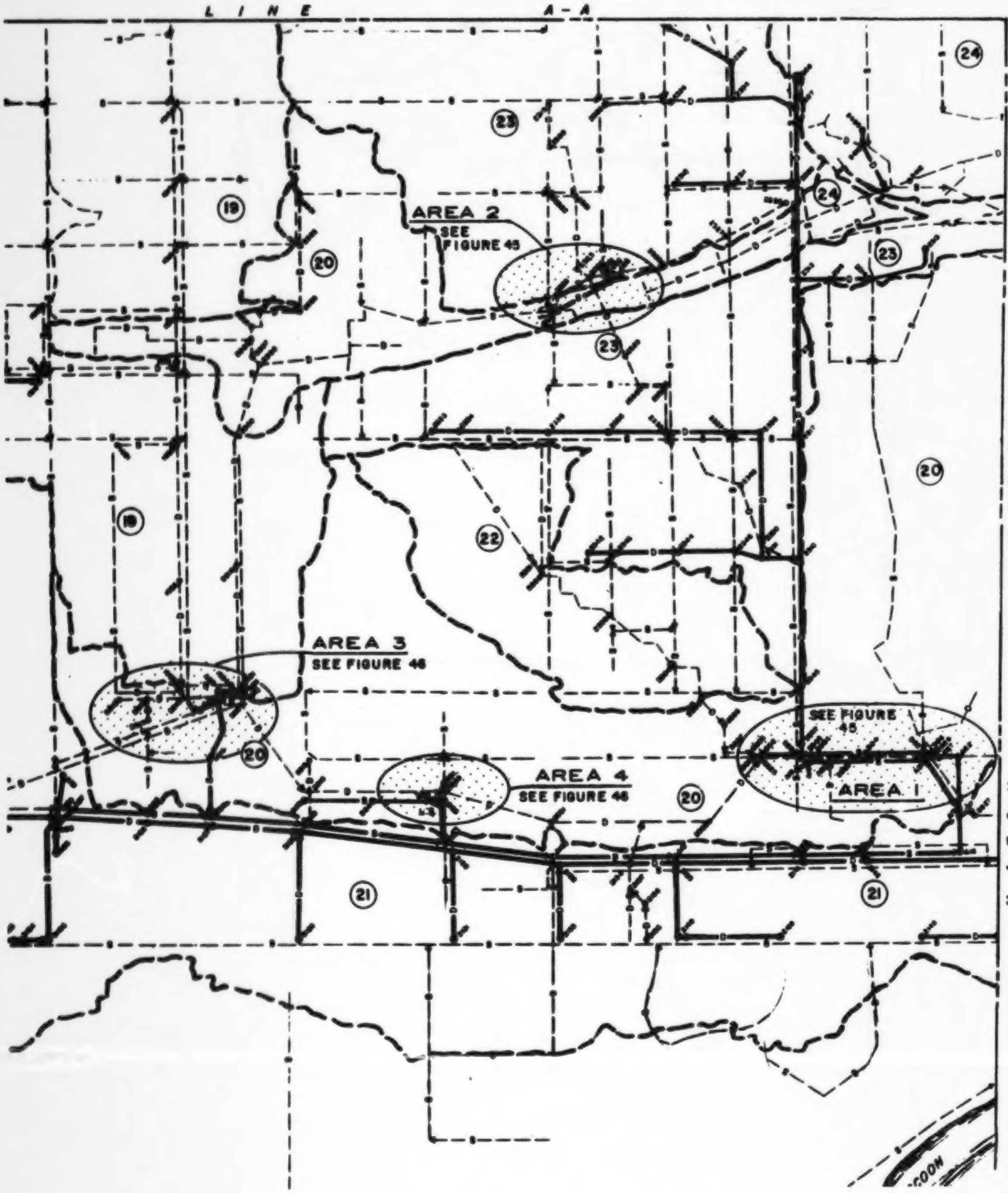

N
SCALE 1" = 800'


4TH ST.
33RD ST
32ND ST.
31ST ST.
30TH ST.
29TH ST.
28TH ST
27 TH ST.
26TH ST
5TH ST
24TH ST.
10
7
11
9
12
8
MATCH LINE B-B
LINE A-A

SEWER SEPARATION PLAN
SHEET 2

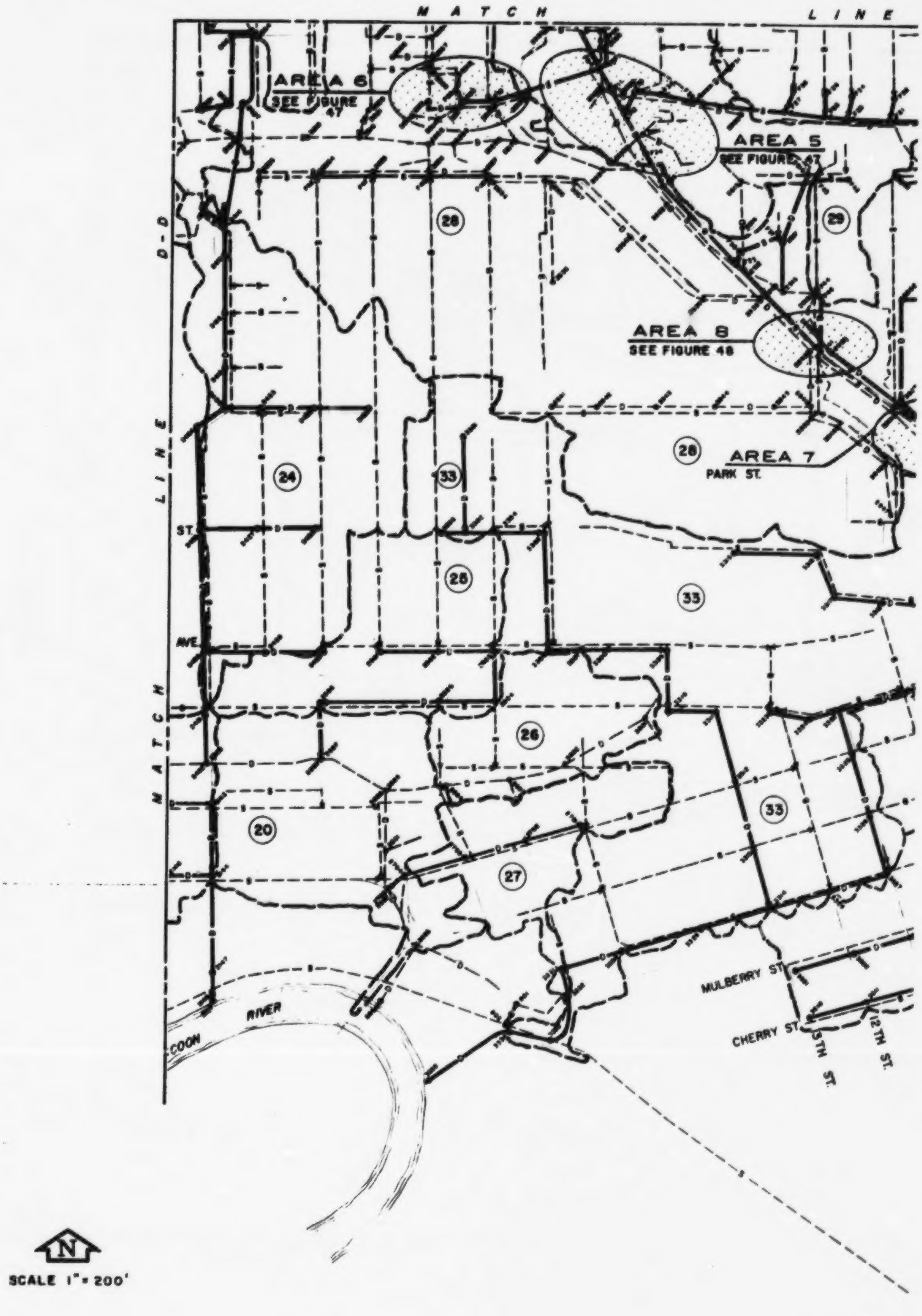
FIGURE 42

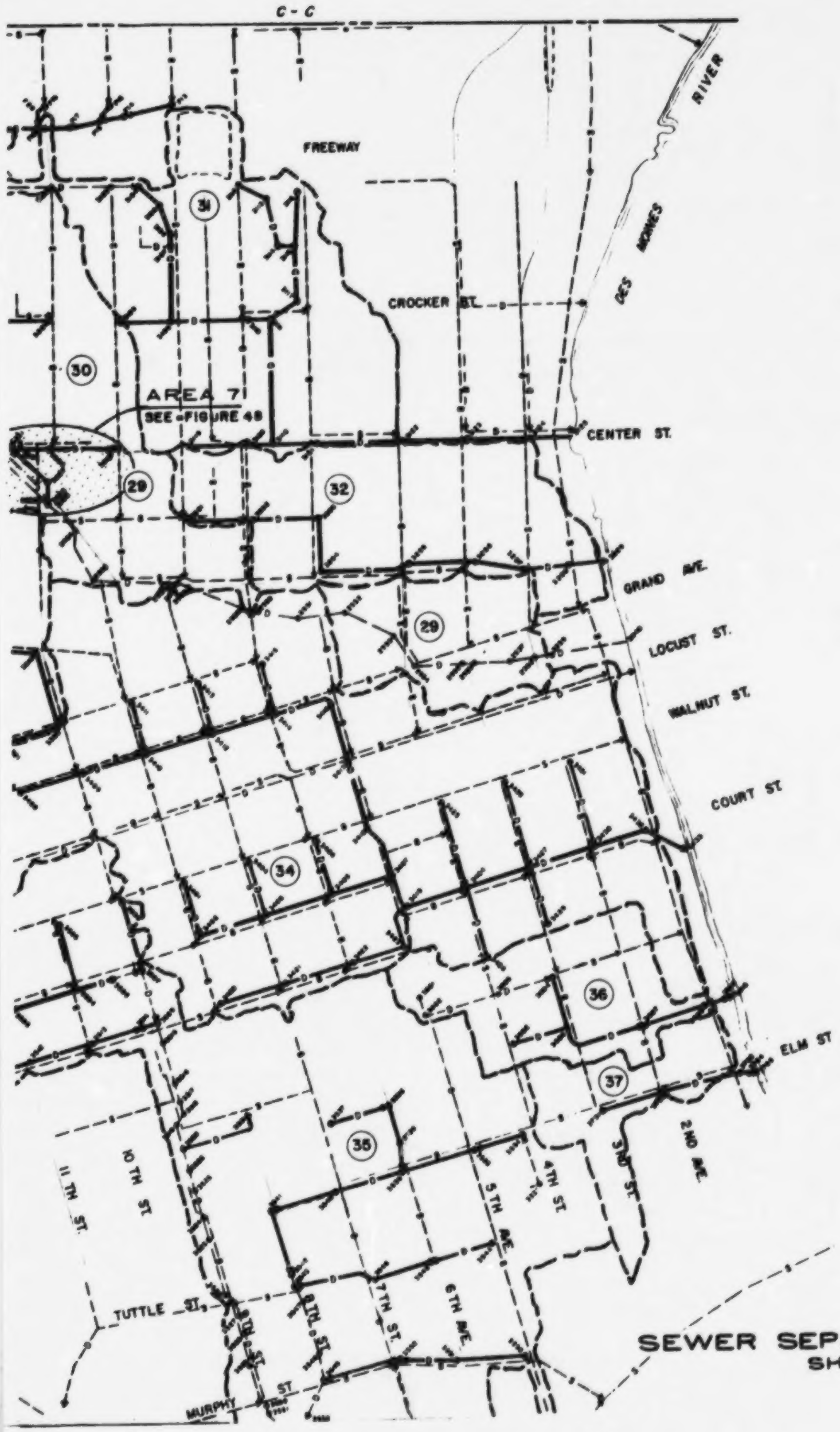

MATCH
KINGMAN BLVD.
OBSERVATORY DR.
CENTER ST.
PLEASANT ST.
HARWOOD DR.
WOODLAND AVE.
GRAND AVE.
⑮
⑯
⑱
⑲
⑰
㉑
N
SCALE 1" = 800'


LINE A-A
AREA 2
SEE FIGURE 45
AREA 3
SEE FIGURE 46
AREA 4
SEE FIGURE 46
SEE FIGURE 45
AREA 1
MATCH LINE D-D
19
20
21
22
23
24
LAGOON

SEWER SEPARATION PLAN
SHEET 3

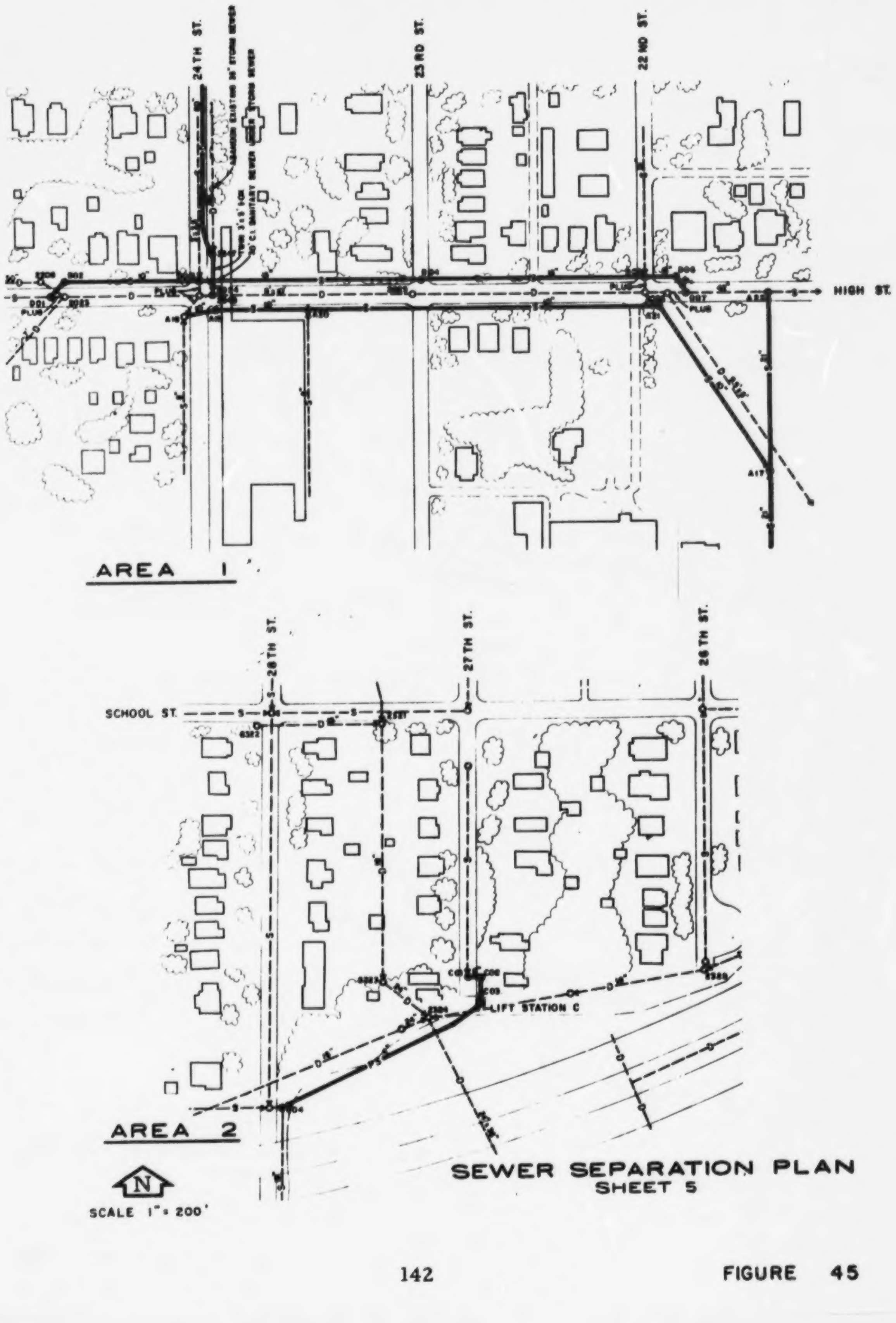
FIGURE 43


MATCH LINE
AREA 6
SEE FIGURE 47
AREA 5
SEE FIGURE 47
28
29
D-D
AREA 8
SEE FIGURE 48
MATCH LINE
24
33
28
AREA 7
PARK ST.
ST.
25
33
AVE.
26
20
33
27
MULBERRY ST.
COON RIVER
CHERRY ST.
13TH ST.
12TH ST.
N
SCALE 1" = 200'


C - C
FREEWAY
DES MOINES RIVER
31
CROCKER ST.
30
AREA 7
SEE FIGURE 48
CENTER ST.
29
32
GRAND AVE.
29
LOCUST ST.
WALNUT ST.
COURT ST.
34
36
ELM ST
37
35
11 TH ST.
10 TH ST
5TH AVE.
4TH ST
3RD ST.
2ND AVE.
TUTTLE ST.
9TH ST.
8TH ST.
7TH ST.
6TH AVE.
MURPHY ST.

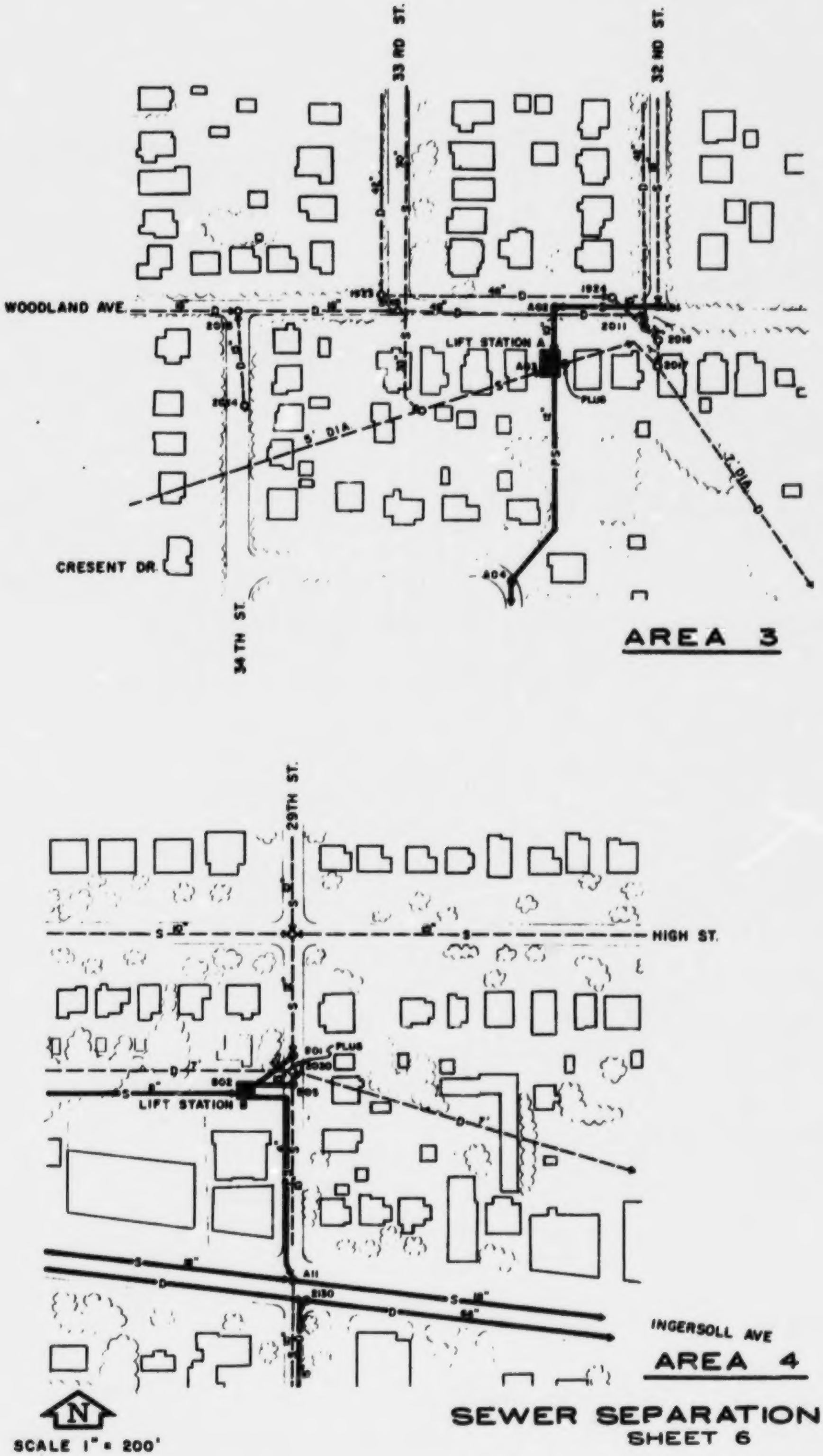
SEWER SEPARATION PLAN
SHEET 4

FIGURE 44


24TH ST.
23RD ST.
22ND ST.
HIGH ST.
PLUG
LIFT STATION C
AREA 1
28TH ST.
27TH ST.
26TH ST.
SCHOOL ST.
AREA 2
N
SCALE 1" = 200'

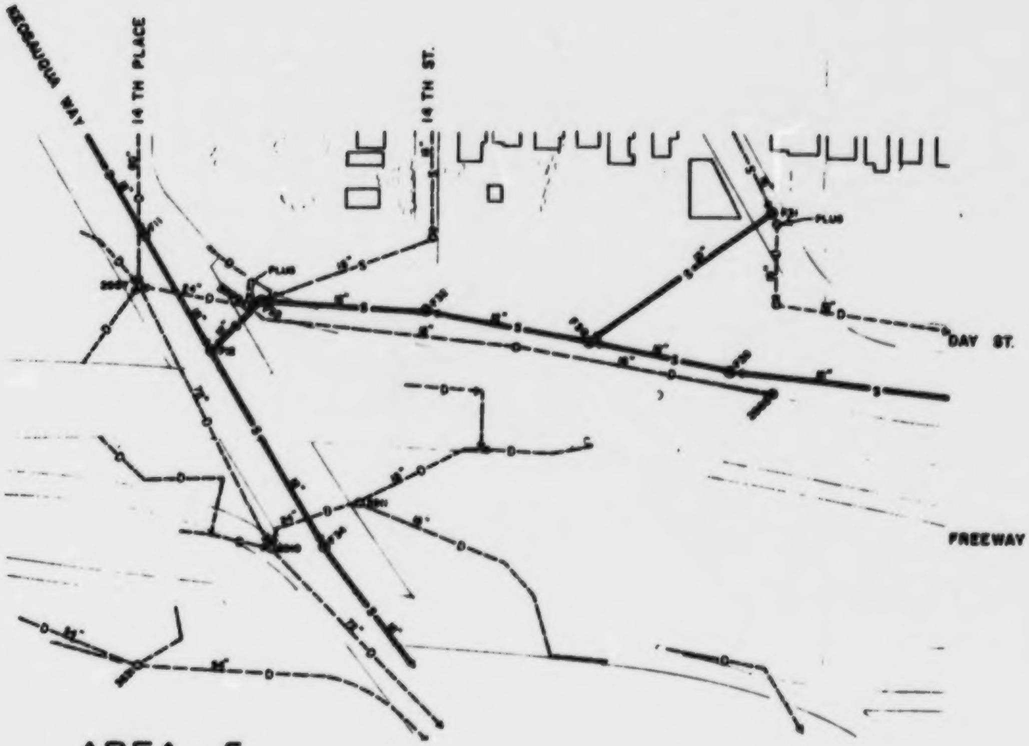
SEWER SEPARATION PLAN
SHEET 5

FIGURE 45

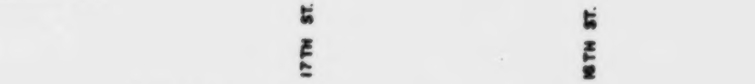

33 RD ST.
32 ND ST.
WOODLAND AVE.
LIFT STATION A
PLUG
5' DIA.
7' DIA.
CRESENT DR.
34TH ST.
AREA 3
29TH ST.
HIGH ST.
PLUG
LIFT STATION B
INGERSOLL AVE
AREA 4
N
SCALE 1" = 200'

SEWER SEPARATION PLAN
SHEET 6

FIGURE 46


KEOSAUQUA WAY
14 TH PLACE
14 TH ST.
PLUG
DAY ST.
FREEWAY

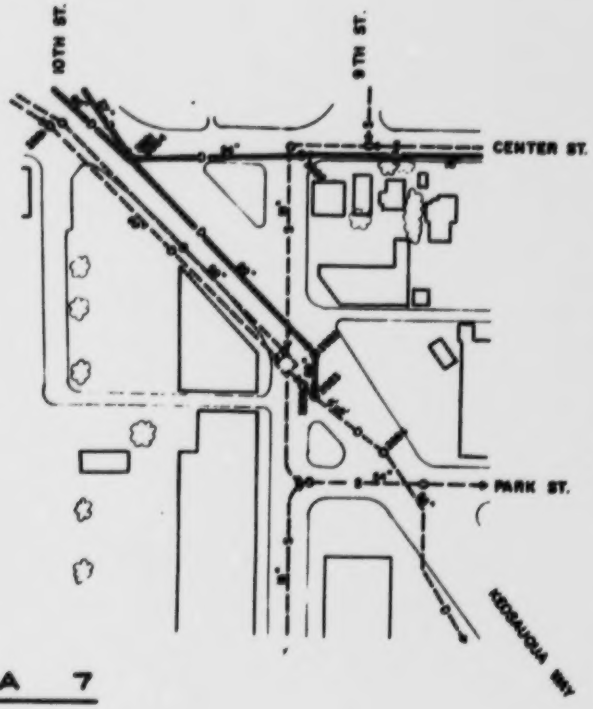
AREA 5


17TH ST.
16TH ST.
PLUG
DAY ST.
FREEWAY
N
SCALE 1" = 200'

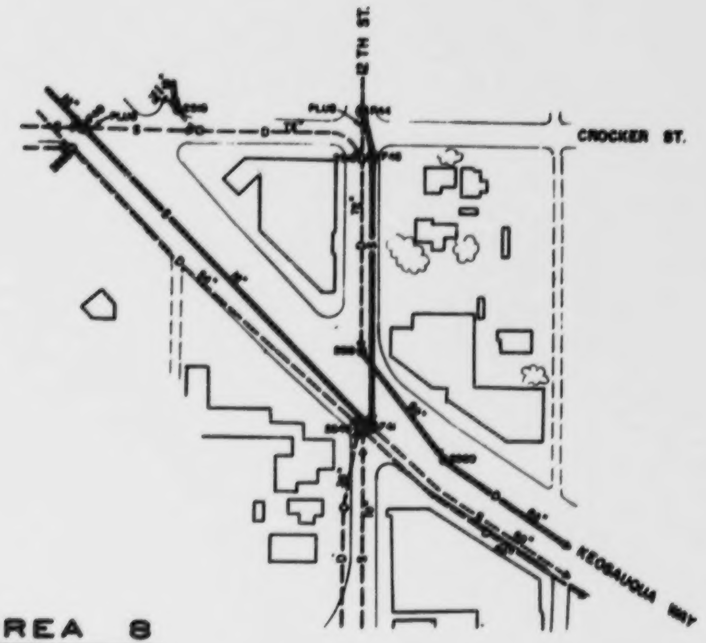
AREA 6

SEWER SEPARATION PLAN
SHEET 7

FIGURE 47


10TH ST.
9TH ST.
CENTER ST.
PARK ST.
KEOSAUQUA WAY

AREA 7


12TH ST.
PLUG
CROCKER ST.
KEOSAUQUA WAY

AREA 8

N

SCALE 1" = 200'

SEWER SEPARATION PLAN
SHEET 8

FIGURE 48

SECTION X

TREATMENT OF COMBINED SEWER OVERFLOWS AND STORM WATER DISCHARGES

GENERAL

To establish the requirements of future wastewater collection and treatment systems, a basis for design of such facilities was prepared. Facilities for elimination or collection and treatment of combined sewer overflows were then designed to be consistent with the envisioned sanitary collection and treatment facilities. The various areas or systems which release combined sewer overflows to the streams are considered herein according to points of concentration for discharge. Alternate solutions are first described and evaluated for these individual systems. Overall schemes for elimination or reduction of combined sewer overflow are then developed as a composite of the facilities proposed for individual systems. The areas which have been investigated include all known combined sewer areas and are grouped into the following systems:

1. Closes Creek System
2. West Side Interceptor
3. Ingersoll Run System
4. East Side System
5. East 18th Street System
6. South Side Trunk System
7. Scott Street Lift Station and Storm Outfall
8. Case Lake Treatment Complex

Concepts for Overflow and Stormwater Discharge Abatement

Three basic concepts for abatement of combined sewer overflows and control of stormwater discharges are considered. The concepts considered do not provide the same degree of pollution abatement, therefore the effectiveness of each relative to reducing urban pollution discharges is discussed. The concepts considered are (1) completely separate all combined sewers, (2) intercept and treat combined sewer overflows, and (3) intercept and treat all combined sewer overflows and urban stormwater discharges.

In all of the concepts, the "wet" dry weather overflow condition is considered to be eliminated by the construction of additional wastewater treatment facilities by the City of Des Moines. The expanded plant will have a maximum hydraulic capacity of 130 MGD and will be capable of

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treating all combined sewer flows carried to the plant by the existing outfall sewers (15). It is assumed that the average daily BOD_5 discharge from the Des Moines plant will meet the State requirements of 7500 pounds per day to the river. Four plans for combined sewer overflow abatement have been developed within the concepts described. They are described as follows:

Plan A is complete separation of all combined sewers. This plan has been described in detail in Section IX of this report. Further discussion of complete separation will be confined to the evaluation of effectiveness at the end of this section.

Plan B-1 is to intercept and treat combined sewer overflows. No overflow from the combined sewer system would be permitted under this plan. The plan includes separation of certain areas where interception and treatment of all combined flows is deemed impractical.

Plan B-2 is to intercept and treat combined sewer overflows as in Plan B-1, however, greater use will be made of existing systems and small quantities of overflow will be permitted during high intensity storms. Only small areas of combined sewers would be separated.

Plan C provides for the treatment of all stormwater discharges in addition to combined sewer overflow abatement as described in Plan B-1. Four areas were selected to develop "typical" stormwater treatment facilities. Based on the analysis of these areas, the costs of treating all urban stormwater discharges will be evaluated.

The improvements proposed for each of the individual systems are described in detail in subsequent parts of this Section.

FUTURE WASTEWATER COLLECTION SYSTEM

A study was made of urban growth as it pertains to future collection systems. Certain assumptions were made as to location of future trunk and relief sewers and the area of service for each system. Reference to Figures 2 and 13 will assist the reader to envision the improvements described and the areas served by each system. The assumptions made are generally in concurrence with the utilities planning report prepared for the Central Iowa Regional Planning Commission (13). This report recommends either (1) a future treatment plant site north of Interstate 35-80 on the east side of the Des Moines River, or (2) a future trunk sewer on the east side of the river which would convey the wastes from the areas north of the Interstate to an expanded plant below Des Moines.

It is assumed that the line on the east side of the river will be constructed at some point in the future and will serve the City of Ankeny, the Urbandale Sanitary District and other areas developing north of Des Moines.

The discharge of treated wastes above Des Moines would be to a section of river that is already heavily used for recreation and will most likely increase in usage due to the new Saylorville Reservoir and the improved river quality therefrom. Two waste treatment facilities presently discharge to streams which reach the Des Moines River immediately above Des Moines; the Urbandale Sanitary District Plant on Beaver Creek and the Ankeny-John Deere Plant on Saylor Creek. Samples were collected from these two streams during the August and October river samplings. Both periods showed a dissolved oxygen depletion on Saylor Creek, presumably the result of waste treatment plant effluent. Development north of Des Moines will undoubtedly be accelerated by the completion of the Saylorville Reservoir, requiring the extension of sewer service to this area in the near future.

In the development of a collection system plan, it was assumed that the existing areas on the west side of the river above the Closes Creek Basin would be siphoned to the new relief line on the east side. This would relieve the existing West Side Interceptor, which is already near capacity due to high infiltration, and would divert a large volume of separate sanitary flow out of the combined system below Closes Creek. As subsequently explained, this is consistent with a general philosophy which Des Moines should adopt regarding extension of their collection system.

A number of alternatives should be investigated for the proposed East Side Relief Sewer, such as tunneling from the Birdland Lift Station to below University Avenue and whether to expand the two existing lift stations for continued use or abandon them for gravity flow. It appears, however, that the most feasible route through the business district would be on the east side, where the relief sewer could be constructed without connection to the existing combined sewer system. This would provide a large capacity separate trunk sewer serving the north side, as the Southwest Outfall does the west side. These separate trunk sewers would then be serving a large percentage of the growth areas and would connect to the main outfall at a point below the major combined sewer areas. Reducing the quantity and pollutional strength of combined sewer overflows depends heavily on the general philosophy of diverting separate sanitary and storm flows from the combined system. Storm runoff from the separated areas would be maintained separate to the point of discharge or treatment, as the case may be. Also the collection system can be designed so as to give separate and high strength sanitary wastes priority to conventional mechanical treatment processes. By this approach, combined sewer flows would be reduced in quantity and pollutional load, and may be intercepted and treated more easily.

The existing Bloomfield system includes the upper reaches of the Yeader Creek Basin which will expand downstream and eventually require a trunk sewer outletting somewhere below the Yeader Creek Lake. It is assumed that such a line would be in existence and tributary to the main outfall by 1990.

The North River Basin is presently served by five separate collection and treatment systems, therefore present flows for that basin are not included. This basin will eventually either be served by a single treatment facility near the mouth of the North River, or the flow will be transported to a downstream treatment facility serving the entire metropolitan area. There will undoubtedly be an interim treatment facility, or facilities, in the Middle Creek and North River Basins. Future flows estimated for this basin are given for purposes of future treatment only, since this separate system would not directly affect overflow problems. Future flows are for the total population of the basin, and do not consider that this may be served by more than one system for an interim period.

BASIS OF DESIGN

Future Sanitary Flows

On the basis of the population studies, the dry weather sanitary wastes sampling, the measurement of wet-season infiltration, and the industrial waste survey conducted by the City of Des Moines, an estimate of present and future sanitary flows has been prepared. This data was used for evaluating the capacity of existing sewers to handle present and future sanitary flows, and for determining existing sewer's reserve capacity for handling combined flows. Using pollutant concentrations obtained from sampling, estimated flows were also used to evaluate the load contributed from each drainage basin to overflow storage and treatment facilities.

Table 16 gives the estimated present, 1990 and 2020 sanitary flows, with contributing populations and the breakdown of domestic, industrial and infiltration flow. Average daily domestic flows were based on 100 gallons per capita per day which was slightly greater than that measured during dry weather sampling, but does not include an allowance for infiltration. Present industrial flows account for only the major water-using industries surveyed to date by the city. Future

TABLE 16
SUMMARY OF SANITARY FLOWS

DESCRIPTION	PRESENT FLOW - MGD						TYPE OF PRESENT SYSTEM
SYSTEM - AREA NO	ESTIMATED POPULATION	DOMESTIC AVE. DAILY	INDUSTRIAL ① AVE. DAILY	INFILTRATION	TOTAL AVE. DAILY	DESIGN FLOW	
WEST SIDE INTERCEPTOR - I-1 ②	79,000	7.90	0.97	10.48	19.35	37.96	COMBINED
EAST SIDE INTERCEPTOR - I-2, III-1, III-2 ②	16,420	1.64	–	2.68	4.32	6.28	COMBINED
SOUTH WEST OUTFALL - II-1, II-2	59,700	5.97	–	6.47	12.44	20.50	SEPARATE
SOUTHERN HILLS TRUNK - II	5,000	0.50	–	0.50	1.00	–	SEPARATE
SCOTT STREET SUBTOTAL	160,120	16.01	0.97	20.13	37.11	62.38	COMBINED
SOUTH SIDE TRUNK - III	15,300	1.53	–	11.30	12.83	6.73	COMBINED
EAST 18TH STREET TRUNK - IV-1, IV-2	17,300	1.73	2.18	3.10	7.01	19.21	COMBINED
MAIN OUTFALL AREA - IX	3,000	0.30	–	0.45	0.75	–	SEPARATE ⑥
BLOOMFIELD SYSTEM - XIII-1 ③	13,400	1.34	–	2.01	3.35	6.03	COMBINED
MAIN OUTFALL @ WWTP SUBTOTAL	209,120	20.91	3.15	36.99	61.05	86.18	COMBINED
FOUR MILE TRUNK - X ⑤	35,600	3.56	1.70	5.32	10.58	–	SEPARATE
YEADER CREEK BASIN - XIII-2 ③	–	–	–	–	–	–	SEPARATE
TOTAL TO WWTP	244,720	24.47	4.85	42.31	71.63	99.31	COMBINED
NORTH RIVER BASIN - X ⑤	10,400	–	–	–	–	–	SEPARATE

NOTES:
① PRESENT INDUSTRIAL FLOW DETERMINED FROM CITY OF DES MOINES SURVEY
② SOUTH SIDE ABOVE CLOSED CREEK AREA TO EAST SIDE FOR FUTURE
③ YEADER CREEK BASIN INCLUDED IN BLOOMFIELD SYSTEM AT PRESENT
④ INCLUDES INDUSTRIAL WASTE FLOWING DIRECTLY TO WWTP
⑤ FIVE SEPARATE SYSTEMS SERVE AREA AT PRESENT
⑥ SEPARATE SANITARY SEWERS TO MAIN OUTFALL FROM THIS AREA

DESCRIPTION	1990 FLOW - MGD						2020 FLOW - MGD					
SYSTEM - AREA NO	ESTIMATED POPULATION	DOMESTIC AVE. DAILY	INDUSTRIAL AVE. DAILY	INFILTRATION	TOTAL AVE. DAILY	DESIGN FLOW	ESTIMATED POPULATION	DOMESTIC AVE. DAILY	INDUSTRIAL AVE. DAILY	INFILTRATION	TOTAL AVE. DAILY	DESIGN FLOW
WEST SIDE INTERCEPTOR - I-1 ②	55,500	5.55	1.25	16.96	23.76	32.30	56,500	5.65	1.90	17.15	24.70	38.58
EAST SIDE INTERCEPTOR - I-2, III-1, III-2 ②	83,500	8.35	0.40	10.83	19.58	29.56	112,500	11.25	0.80	12.87	24.92	38.37
SOUTH WEST OUTFALL - II-1, II-2	61,000	6.10	1.30	11.35	18.75	30.00	112,000	11.20	2.00	14.56	27.76	43.95
SOUTHERN HILLS TRUNK - II	10,000	1.00	–	1.00	2.00	–	25,000	2.50	–	2.50	5.00	–
SCOTT STREET SUBTOTAL	210,000	21.00	2.95	51.04	67.00	96.42	306,000	30.60	4.70	48.08	83.96	121.25
SOUTH SIDE TRUNK - III	10,000	1.00	–	2.70	3.70	7.75	20,000	2.00	–	3.00	5.00	8.50
EAST 18TH STREET TRUNK - IV-1, IV-2	19,000	1.90	4.35	2.65	8.90	18.57	20,000	2.00	6.70	3.00	11.70	25.25
MAIN OUTFALL AREA - IX	3,000	0.30	0.80	0.15	1.25	–	3,000	0.30	1.30	0.45	2.05	–
BLOOMFIELD SYSTEM - XIII-1 ③	5,000	0.50	–	0.75	1.25	–	6,000	0.60	–	0.90	1.50	–
MAIN OUTFALL @ WWTP SUBTOTAL	247,000	24.70	8.10	57.75	90.55	135.05	355,000	35.50	12.70	55.01	103.21	152.16
FOUR MILE TRUNK - X ⑤	60,000	6.00	3.15	6.10	15.25	–	95,000	9.50	5.00	11.20	25.70	–
YEADER CREEK BASIN - XIII-2 ③	15,000	1.50	–	[illegible]	3.50	–	20,000	2.00	–	3.00	5.00	–
TOTAL TO WWTP	322,000	32.20	11.25	66.00	109.45	139.00	470,000	47.00	17.70	69.21	133.91	175.31
NORTH RIVER BASIN - X ⑤	20,000	2.00	–	3.12	5.12	–	40,000	4.00	–	4.52	8.52	–

industrial flows include a growth allowance for existing industries plus an allowance for new industries which may locate in the Des Moines area. The infiltration allowance used, based on the results of the wet-season measurements, was generally 150 gallons per capita per day for existing developed areas and 100 gallons per capita per day for the growth areas. Existing infiltration allowances varied from the 150 gallons per capita figure in a very few cases, based on specific knowledge of the watershed or actual measurement.

The infiltration allowance is expressed in gallons per capita per day instead of the commonly used design parameter of gpd per mile. This was due to time limitations on relating measured infiltration to length of sewer and the fact that footing drains were believed to be contributing a major portion of the infiltration flow.

Design flows are the maximum instantaneous flow expected and were determined as follows. For domestic peak flows the formula $Q = q\left(1 + \frac{14}{4 + p^{0.5}}\right)$ was used, where Q = peak flow rate, q = average daily flow, and p = population served in 1,000's. This formula describes curves published in the Water Pollution Control Federation's Manual of Practice No. 9, "Design and Construction of Sanitary and Storm Sewers" (14). The ratio of peak to average daily flow determined by the formula was limited to a maximum of 400 percent and a minimum of 200 percent for the Des Moines system.

Industrial peak flows were assumed to be 250 percent of average daily flow in all sewers except the inlet to the treatment plant where 200 percent of average daily flow was used. Infiltration flows were considered to be at a constant rate.

Peak Combined Flows

Pipe and conduit sizes proposed were determined by the following criteria:

Storm Flows. Rainfall Intensity-Duration-Frequency Curves for a 10-year return period in Des Moines, Iowa were used. In nearly all cases the existing combined systems will carry only a portion of a 10-year rainfall, therefore it is assumed that during peak flow periods only a small portion of the precipitation enters the systems. In some locations, combined sewage may leave the system and return to the surface due to lack of capacity in the pipes. This source of pollution cannot be eliminated without completely rebuilding large portions of the combined systems. An exact analysis of the combined sewer overload could not be made without very exhaustive field surveys and hydralic studies. In most cases the quantity of storm flow at the juncture of the combined systems and the interceptor was based on the estimated pipe capacity upstream minus the average daily sanitary flow.

Sanitary Flows. Existing sanitary drainage basins or subbasins contributing to combined sewer systems are considered for ultimate development if the basin will become fully developed within the design period. Population estimates are based on 15 persons per acre for ultimate development and a reasonable projection for the design period if ultimate development will not occur. Sanitary flows are based on 100 gallons per capita per day, and peak daily flow in trunk and interceptor sewers was estimated to be 2.5 times the average daily flow.

Quantitative Flows. Volumetric storage capacities for ponds and impoundments are based on the following criteria:

Storm Flows. The design rainstorm selected produces 6 inches of precipitation within a period of 24 hours. The rate of precipitation was assumed to be in direct proportion to the rainfall intensities for a 10-year return period. Volumetric design curves are shown in Figure 49. An illustration of the method used to determine quantity of storm and sanitary flows is shown in Figure 50.

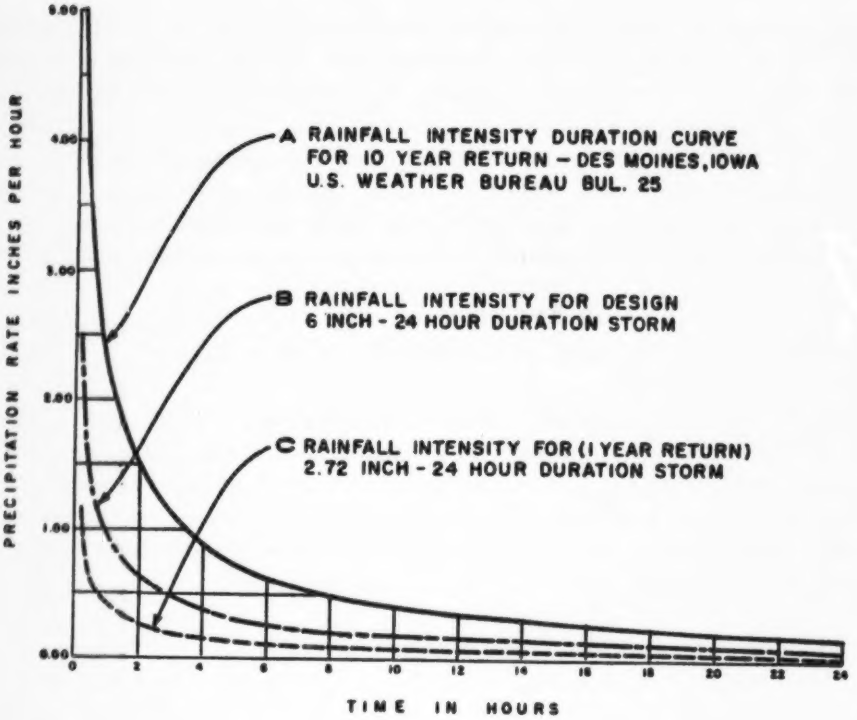
The average annual maximum precipitation for a 24-hour period was also evaluated to obtain an estimate of the annual maximum 24-hour storm. Weather Bureau records of the annual maximum 24-hour precipitation for the past 18 years were used to obtain an average value of 2.72 inches in a 24-hour period. The rate of precipitation for the 2.72-inch rain was assumed to be in direct proportion to the volumetric design curve for the 6-inch rain.

Sanitary Flows. That portion of the total combined flow consisting of sanitary sewage flow was computed on the basis of average daily flows of 100 gallons per capita per day for the area served.

Quantity of Combined Sewage Overflow

For interception and treatment of combined sewage overflows two alternative plans have been developed. Plan B-1 consists of complete elimination of combined sewage overflows, while Plan B-2 allows a minor quantity of overflow to enter the river during extreme peak flows. Where the cost of separating storm and sanitary systems would be minimal, that method was used to reduce the quantity of overflow.

Flows from the known combined sewer areas are listed in summary form in Table 17. The table contains a breakdown of the storm and sanitary

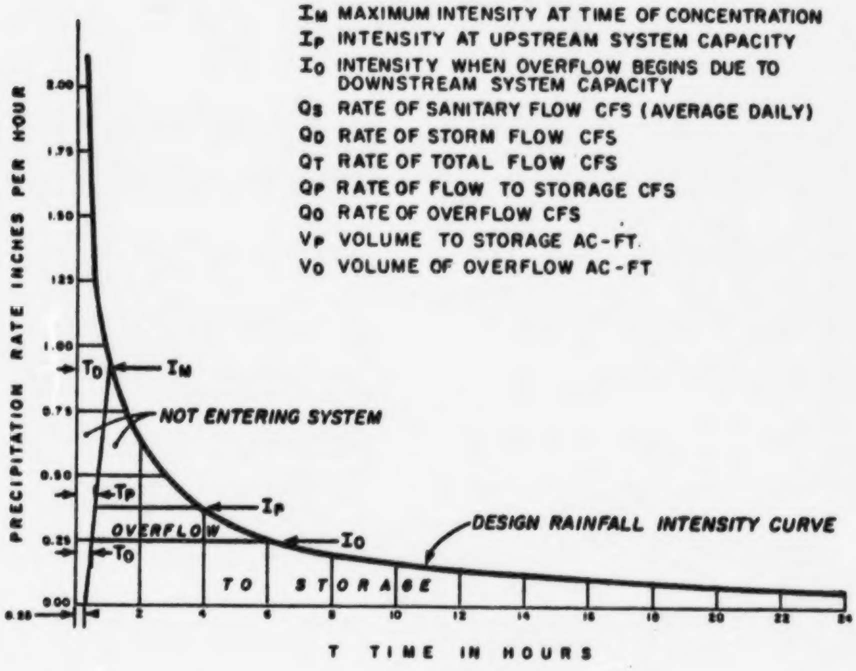


1. CURVE A - PRODUCES TOTAL PRECIPITATION OF 14.07 INCHES IN 24 HOURS
2. CURVE B - PRODUCES TOTAL PRECIPITATION OF 6.00 INCHES IN 24 HOURS. INTENSITIES ARE 0.4264 TIMES CURVE A INTENSITIES.
3. CURVE C - PRODUCES TOTAL PRECIPITATION OF 2.72 INCHES IN 24 HOURS. INTENSITIES ARE 0.1933 TIMES CURVE A INTENSITIES.

RAINSTORM VOLUMETRIC DESIGN CURVES

FIGURE 49

A	STORM DRAINAGE AREA ACRES
C	RUNOFF FACTOR
T_D	TIME OF CONCENTRATION AT POINT CONSIDERED
T_P	TIME UPSTREAM SYSTEM REACHES CAPACITY
T_O	TIME OVERFLOW BEGINS
ΔT	INCREMENT OF TIME
I_M	MAXIMUM INTENSITY AT TIME OF CONCENTRATION
I_P	INTENSITY AT UPSTREAM SYSTEM CAPACITY
I_O	INTENSITY WHEN OVERFLOW BEGINS DUE TO DOWNSTREAM SYSTEM CAPACITY
Q_S	RATE OF SANITARY FLOW CFS (AVERAGE DAILY)
Q_D	RATE OF STORM FLOW CFS
Q_T	RATE OF TOTAL FLOW CFS
Q_P	RATE OF FLOW TO STORAGE CFS
Q_O	RATE OF OVERFLOW CFS
V_P	VOLUME TO STORAGE AC-FT.
V_O	VOLUME OF OVERFLOW AC-FT



FORMULAE

I_M FROM CURVE FOR GIVEN T_D

$$I_P = \frac{\text{UPSTREAM LINE CAPACITY (CFS)} - Q_S}{CA}$$

$$I_O = \frac{Q_T - \text{DOWNSTREAM LINE CAPACITY (CFS)}}{CA}$$

$$T_O = 0.25 - \frac{I_O}{I_M}(T_D - 0.25)$$

$$T_P = 0.25 - \frac{I_P}{I_M}(T_D - 0.25)$$

$Q_D = CIA$ UPPER LIMIT $I = I_P$

$Q_T = Q_D + Q_S$

$Q_P = Q_T$ Q_P MAX. = CAP. DOWNSTREAM LINE

$Q_O = Q_T -$ DOWNSTREAM LINE CAPACITY

$V_P = (\text{AVE.})\ Q_P \cdot \Delta_T \cdot 0.08264$

$V_O = (\text{AVE.})\ Q_O \cdot \Delta_T \cdot 0.08264$

TABLE OF COMPUTATIONS

T HRS	Δ_T HRS	Q_D CFS	Q_T CFS	Q_P CFS	V_P AC-FT.	ACC. V_P AC-FT	Q_O CFS	V_O AC-FT	ACC. V_O AC-FT.

VOLUMETRIC ANALYSIS
COMBINED SEWAGE SYSTEMS

FIGURE 50

TABLE 17

SUMMARY OF COMBINED SEWAGE & OVERFLOW QUANTITIES

PLAN B-1

SYSTEM	AD ACRES	QS CFS	6" 24 HOUR RAINSTORM IP IN/HR	VP AC. FT.	VPS AC. FT.	VPD AC. FT.	VO AC. FT.	VOS AC. FT.	2.72" 24 HOUR RAINSTORM I MAX IN/HR	VP AC. FT.	VPS AC. FT.	VPD AC. FT.	VO AC. FT.	VOS AC. FT.
BEAVER AVE.	0.00	3.81	0.00	7.84	7.84	0.00	0.00	0.00	0.00	7.84	7.84	0.00	0.00	0.00
CLOSES CREEK	1493.83	3.67	0.00	433.01	7.27	425.74	0.00	0.00	0.00	200.27	7.27	193.00	0.00	0.00
HICKMAN-CHAUTAUQUA	0.00	0.07	0.00	0.14	0.14	0.00	0.00	0.00	0.00	0.14	0.14	0.00	0.00	0.00
FRANKLIN-ARLINGTON	112.88	0.73	0.71	29.31	1.41	27.90	4.27	0.00	0.54	16.01	1.41	14.60	0.00	0.00
COLLEGE-ARLINGTON	106.06	0.41	0.79	42.34	0.81	41.53	5.81	0.00	0.88	22.30	0.81	21.49	0.00	0.00
INDIANA 2ND AVE.	17.70	0.06	1.46	4.88	0.10	4.78	0.27	0.00	0.77	2.39	0.10	2.29	0.00	0.00
FRANKLIN-UNIVERSITY	63.90	0.40	0.74	16.77	0.79	15.98	2.23	0.00	0.77	8.57	0.79	7.78	0.48	0.00
KEO-GRAND @ 1ST & GRAND	563.88	1.80	0.64	104.59	3.72	100.87	25.88	0.00	0.32	61.15	3.72	57.43	0.00	0.00
WALNUT @ 1ST	436.38	3.99	0.60	85.39	7.90	77.49	43.88	0.00	0.39	62.44	7.90	54.54	0.00	0.00
COURT STREET @ 1ST	75.86	0.23	0.84	23.95	0.46	23.49	2.00	0.00	0.58	12.53	0.46	12.07	0.00	0.00
ELM STREET @ 1ST	86.10	0.33	0.80	25.26	0.65	24.61	2.80	0.00	0.58	13.10	0.65	12.45	0.00	0.00
INGERSOLL RUN	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
BIRDLAND LIFT TO GRAND	296.07	3.20	0.19	53.51	6.34	47.17	36.90	0.00	0.38	34.82	6.34	28.48	9.69	0.00
BELOW BIRDLAND TO 1ST @ MAPLE	10.58	0.23	4.33	3.47	0.46	3.01	0.00	0.00	0.57	1.83	0.46	1.37	0.00	0.00
E. DES MOINES AVE. @ E. 1ST.	0.00	0.06	0.00	0.12	0.12	0.00	0.00	0.00	0.00	0.12	0.12	0.00	0.00	0.00
E. GRAND AVE. & E. 1ST	116.66	0.43	2.40	34.10	0.85	33.25	0.00	0.00	0.36	15.95	0.85	15.10	0.00	0.00
E. LOCUST & E.1ST	27.70	0.08	2.31	8.02	0.12	7.90	0.00	0.00	0.35	3.70	0.12	3.58	0.00	0.00
E. WALNUT & E. 1ST	45.20	0.11	1.96	13.12	0.22	12.90	0.00	0.00	0.34	6.08	0.22	5.86	0.00	0.00
E. COURT ST. & E. 1ST	48.20	0.13	0.73	14.01	0.28	13.73	0.00	0.00	0.33	6.51	0.28	6.23	0.00	0.00
S.E. 9TH ST. @ SIPHON	135.67	4.01	0.18	27.02	7.94	19.08	19.52	0.00	0.36	20.74	7.94	12.80	4.72	0.00
E. 18TH ST. & COURT AVE.	150.30	1.13	3.82	38.82	2.24	36.58	0.00	0.00	0.39	19.03	2.24	16.79	0.00	0.00
REMAINING DRAINAGE AREA	45,457.82						12,034.39						5,464.28	
TOTALS	49,184.00	24.93		965.47	49.36	916.11	12,178.69	0.00		515.18	49.34	465.84	5,479.17	0.00

PLAN B-2

SYSTEM	AD ACRES	QS CFS	6" 24 HOUR RAINSTORM IP IN/HR	VP AC. FT.	VPS AC. FT.	VPD AC. FT.	VO AC. FT.	VOS AC. FT.	2.72" 24 HOUR RAINSTORM I MAX IN/HR	VP AC. FT.	VPS AC. FT.	VPD AC. FT.	VO AC. FT.	VOS AC. FT.
BEAVER AVE.	0.00	3.81	0.00	7.84	7.84	0.00	0.00	0.00	0.00	7.84	7.84	0.00	0.00	0.00
CLOSES CREEK	106.83	3.67	0.38	23.45	7.27	16.18	14.26	0.00	0.36	18.23	7.27	10.96	2.84	0.00
HICKMAN-CHAUTAUQUA	0.00	0.07	0.00	0.14	0.14	0.00	0.00	0.00	0.00	0.14	0.14	0.00	0.00	0.00
FRANKLIN-ARLINGTON	112.88	0.73	0.71	29.31	1.41	27.90	4.27	0.00	0.54	16.01	1.41	14.60	0.00	0.00
COLLEGE-ARLINGTON	106.06	0.41	0.79	42.34	0.81	41.53	5.81	0.00	0.88	22.30	0.81	21.49	0.00	0.00
INDIANA-2ND AVE.	17.70	0.06	1.46	4.88	0.10	4.78	0.27	0.00	0.77	2.39	0.10	2.29	0.00	0.00
FRANKLIN-UNIVERSITY	63.90	0.40	0.75	16.77	0.79	15.98	2.23	0.00	0.77	8.57	0.79	7.78	0.48	0.00
KEO-GRAND @ 1ST & GRAND	563.88	1.80	0.64	104.59	3.72	100.87	25.53	0.00	0.32	61.15	3.72	57.43	0.00	0.00
WALNUT @ 1ST	330.39	3.49	0.70	85.39	6.91	78.48	92.88	6.69	0.39	49.66	6.91	42.75	36.02	3.04
COURT ST. @ 1ST	75.86	0.23	0.84	23.95	0.46	23.49	20.58	0.46	0.58	12.53	0.46	12.07	11.12	0.21
ELM ST. & 1ST	86.10	0.33	0.80	25.26	0.65	24.61	27.41	0.65	0.58	13.10	0.65	12.45	11.18	0.29
INGERSOLL RUN	929.19	2.68	0.18	72.67	3.10	69.57	151.61	1.91	0.48	64.70	4.35	60.35	38.86	0.66
BIRDLAND LIFT TO GRAND	296.07	3.20	0.19	53.51	6.34	47.17	36.90	0.00	0.38	34.82	6.34	28.48	9.69	0.00
BELOW BIRDLAND TO 1ST & MAPLE	10.58	0.23	4.33	3.47	0.46	3.01	0.00	0.00	0.57	1.83	0.46	1.37	0.00	0.00
E. DES MOINES AVE. & E.1ST	0.00	0.06	0.00	0.12	0.12	0.00	0.00	0.00	0.00	0.12	0.12	0.00	0.00	0.00
E. GRAND AVE. & E. 1ST	116.66	0.43	2.40	34.10	0.85	33.25	0.00	0.00	0.36	15.95	0.85	15.10	0.00	0.00
E. LOCUST & E. 1ST	27.70	0.08	2.31	8.02	0.12	7.90	0.00	0.00	0.35	3.70	0.12	3.58	0.00	0.00
E. WALNUT & E. 1ST	45.20	0.11	1.96	13.12	0.22	12.90	0.00	0.00	0.34	6.08	0.22	5.86	0.00	0.00
E. COURT & E. 1ST.	48.20	0.13	0.73	14.01	0.28	13.73	0.00	0.00	0.33	6.51	0.28	6.23	0.00	0.00
S.E. 9TH @ SIPHON	135.67	4.01	0.18	27.02	7.94	19.08	19.52	0.00	0.36	20.74	7.94	12.80	4.72	0.00
E. 18TH ST. & COURT AVE.	150.30	1.13	3.82	38.82	2.24	36.58	0.00	0.00	0.39	19.03	2.24	16.79	0.00	0.00
REMAINING DRAINAGE AREA	48,021.63						12,425.84						5,641.33	
TOTALS	49,184.00	28.96		828.58	51.47	777.11	12,832.91	9.71		385.08	52.70	332.38	5,756.24	4.20

DESCRIPTION OF TERMS

AD - COMBINED SEWER STORM DRAINAGE AREA

QS - AVERAGE DAILY FLOW OF SANITARY SEWAGE

IP - MAX. PERCIPITATION RATE, INCHES PER HR., ENTERING SYSTEM

VP - VOLUME ENTERING SYSTEM IN 24 HRS. (TOTAL SANITARY PLUS STORM FLOWS)

VPS - SANITARY PORTION OF VP

VPD - STORM PORTION OF VP

VO - (TOTAL RUNOFF + QS)-VP

VOS - VOLUME OF COMBINED SEWAGE OVERFLOW TO RIVER

I MAX - MAXIMUM RAINFALL INTENSITY FOR TIME OF CONCENTRATION

REMAINING DRAINAGE AREA - SEPARATE SEWERS flows and the quantity of combined sewer overflow from each area. Both the 6-inch and 2.72-inch design rainfalls are listed for the two Plan B alternatives.

Two rainfall intensity values are given: IP and I Max. IP is the intensity in inches per hour which will produce a runoff equal to the capacity of the sewer in question, and is determined by the equation given in Figure 50. I Max. is the maximum intensity from the design curve for the time of concentration to the point in question. The value given in the table applies to the 2.72-inch rainfall only.

Combined sewage overflowing to the river, VOS in the table, is zero for Plan B-1. In Plan B-2 the combined sewage overflow is 9.71 acre-feet for the design 6-inch rainfall and 4.20 acre-feet for the 2.72-inch 24-hour rainfall.

Quantity of Storm Water Discharge

Storm water discharge quantities have been developed for all areas having separated systems. For Plans A and B, separate storm water is discharged untreated to the streams; for Plan C, storm water discharge quantities provide the basis of design for treatment facilities.

All rainfalls producing runoff were included in the analyses. The Rational Formula was used for determining peak flows. Volumetrically, all holding ponds are designed to contain the 6-inch rainfall. Coefficients of runoff were based on rainfall-runoff relationships established in this study. The rainfall-runoff study, described in Section VII, also provided data on the intensity of high frequency storms. An analysis of 126 intense storms occurring from 1951 to July, 1967 produced the intensity-duration-frequency relationships shown in Table 18.

A detailed basis of design is described for each of the facilities evaluated.

TABLE 18

RAINFALL INTENSITIES FOR INTENSE STORMS

Rainfall Intensity in Inches Per Hour

	Return Period - Years			
Duration	2	1	0.5	0.2
5 min.	5.3	4.6	3.8	2.7
10 min.	4.1	3.5	2.9	1.9
20 min.	2.9	2.4	2.0	1.3
30 min.	2.3	1.9	1.5	0.98
1 Hr.	1.4	1.2	0.91	0.59
2 Hr.	0.83	0.70	0.53	0.33

COMBINED SEWER OVERFLOW ABATEMENT SYSTEMS

Individual systems, or areas served by a system of overflow interceptors and treatment facilities, are described herein. The methods considered for elimination of overflows are discussed, and illustrations, design criteria, construction costs, and operation and maintenance costs are given for each alternate.

The "Location Plat," Figure 57 at back of this discussion, shows the relationship of each facility to the overall Plan.

Two of the proposed schemes, Plans B-1 and B-2, deal with the interception and treatment of combined sewer overflows.

Plan B-1

In Plan B-1, no overflow from the combined sewer systems would be permitted. The plan includes separation of certain areas where interception and treatment are deemed impractical. The improvements proposed for each of the individual systems in Plan B-1 are briefly described as follows:

(1) The Closes Creek System includes the construction of two retardation basins for storm runoff containing a small percentage of combined sewer overflow, separation of two small isolated areas, and the Prospect Road Impoundment for storage of overflows.

(2) The West Side Interceptor System includes separation of a small area along Interstate 235 and enlargement of the West Side Storm Interceptor Box.

(3) The Ingersoll Run System, above the present overflow structure, will be separated entirely.

(4) The East Side System includes construction of a 48-inch interceptor sewer to transport combined flows from the northern part of the basin and modifications and extension of the East Side Storm Interceptor Box.

(5) The East 18th Street System consists of diverting combined flows to the Dean Impoundment for treatment.

(6) The South Side Trunk System improvements consist of a lift station for pumping combined sewer overflows into a storm outfall for subsequent treatment at the Case Lake Treatment complex.

(7) The Scott Street Lift Station and Storm Outfall will transport combined sewer overflows from the central part of the city to the Case Lake TreatmentComplex.

(8) The Case Lake Treatment Complex, located across the river from the existing Des Moines Wastewater Treatment Plant, will provide treatment to all combined sewer overflows not treated elsewhere in the individual systems.

Plan B-2

In Plan B-2 small quantities of overflow will be permitted during high intensity storms and greater use will be made of existing facilities than is proposed for Plan B-1. Only small areas of combined sewers will be separated. The improvements proposed for each of the individual systems in Plan B-2 are briefly described as follows:

(1) The Closes Creek System improvements would consist of separating all the combined sewers in this basin (approximately 107 acres) plus two additional small areas and construction of the Prospect Road Impoundment to serve the reduced combined sewer overflow.

(2) The West Side Interceptor System would include separation of a small area along Interstate 235 and minor improvements to the West Side Storm Interceptor Box to permit it to be used as is.

(3) The Ingersoll Run System outlet sewer would be provided with a diversion structure to divert approximately 60 percent of the annual overflow to the Southwest Outfall and ultimately to the Case Lake Treatment Complex.

(4) The East Side System improvements would consist of a retardation basin which would return overflow to the system and modification and extension of the East Side Storm Interceptor Box.

(5) The East 18th Street System consists of diverting overflows to the Dean Lake Impoundment for treatment as in Plan B-1.

(6) The South Side Trunk System improvements would consist of an overflow pumping station as in Plan B-1.

(7) The Scott Street Lift Station and Storm Outfall will transport overflows to the Case Lake Treatment Complex as in Plan B-1; however the size of the facilities will be reduced.

(8) The Case Lake Treatment Complex will be constructed as in Plan B-1; however the size of the facility will be reduced.

The Closes Creek System

This system is located in the northwest sector of the City. The area involved is that served by the separate and combined systems tributary to the West Side Interceptor Sewer above 2nd and Franklin Avenues, and consists of approximately 4050 acres. Combined sewers presently drain 406 acres to this system. The present population of the area is estimated to be 44,000. Land use is primarily residential with light commercial in neighborhood shopping centers.

Closes Creek Retardation Basins. The Closes Creek watershed contains approximately 107 acres served by combined sewers. Overflow from these sewers discharge to Closes Creek. In Plan B-1 two impoundments would be used to hold storm runoff containing these overflows for a period of time, with subsequent release to the Des Moines River at a controlled rate. Dams would be constructed on the two branches of Closes Creek immediately south of Hickman Road at 24th and 27th Streets. These are shown on the location plat and in Figure 51. The larger of the two impoundments, at 27th and Hickman, will serve a drainage area of 1104 acres and have an operating volume of 164 acre-feet between elevations 90.0 and 100.0. The height of the embankment is 52 feet and a spillway capacity of 1,400 cfs is provided to protect the embankment and property downstream. A 42-inch discharge line will permit maximum release of the operating volume within 24 hours and release of the entire volume with 48 hours. The smaller of the two impoundments will serve a drainage area of 283 acres and have an operating volume of 44 acre-feet between elevations 80.0 and 86.0. A spillway capacity of 750 cfs is provided to protect the embankment and the property downstream. The height of embankment is 43 feet. A 27-inch discharge line will provide for maximum release of the operating volume within 24 hours, and release of the entire volume within 48 hours. Both impoundments would have intake structures for controlled drawoff at selected levels. The impoundment areas would be fenced to protect the public.

Each impoundment has capacity in the operating range to store runoff from approximately a 3-inch rain, or will provide a detention time of 1 day in the large impoundment and 2 days in the small one at the low operating level. High runoffs up to the 3-inch rain can be stored for a period of 5 to 7 days to reduce the pollutional load, then be discharged to the river at a controlled rate. Rainfall accumulations of greater than 3 inches which occur during the holding period may overflow the impoundment, depending upon the duration and intensity.

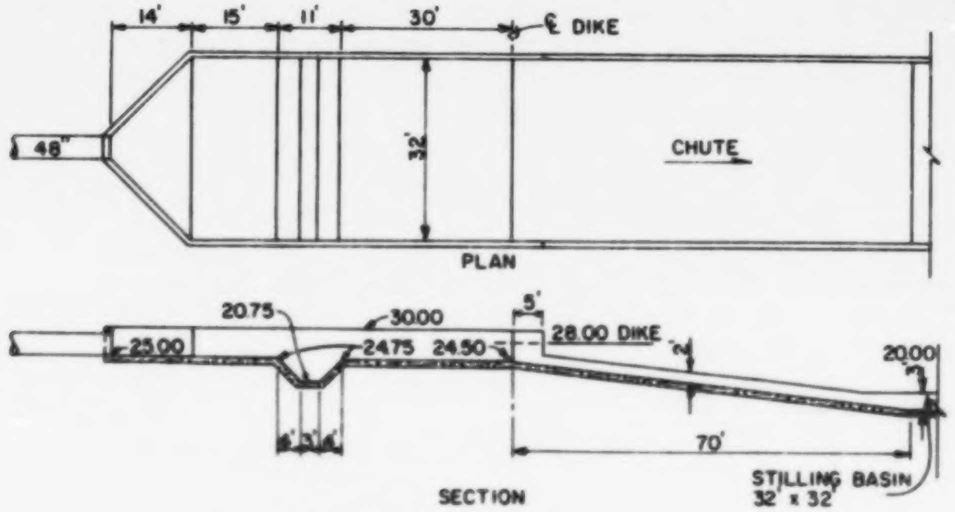
Even so, a 6-inch volumetric, 24-hour rainfall would have a flow-through time of approximately 24 hours in the large basin and 36 hours in the small basin. Based on the monitoring of this watershed at Station 0-2, it was determined that BOD strengths of about 30 mg/l can be expected from the 3-inch rain and 400 to 600 mg/l of total suspended solids. The average of 19 analyses showed suspended solids to be about 20 percent volatile. Based on studies of treatment of storm runoff by Evans, et al. (16), BOD and suspended solids reductions of 35 and 75 percent, respectively, would be conservative for 24-hour detention. The quality of the retardation basin effluents after 24-hours detention of the 3-inch rain should then be in the neighborhood of 20 mg/l of BOD and 100 to 150 mg/l of suspended solids. For smaller rainfalls and runoffs that are stored for a period of several days, the effluent quality would be expected to be proportionately better.

An analysis of oxygen demands in the runoff indicates that the impoundments will remain aerobic without the need for mechanical aeration. No odor problems should be encountered from aerobic basins. Sediment will accumulate in the basins and will have to be removed eventually. Due to the depth and size of the impoundments cleaning will be infrequent and 10-year intervals would be adequate. No lining will be required to prevent seepage. The sides of the basins would be seeded to grass to prevent erosion. Mowing and other routine maintenance will be required to maintain the sites in good condition.

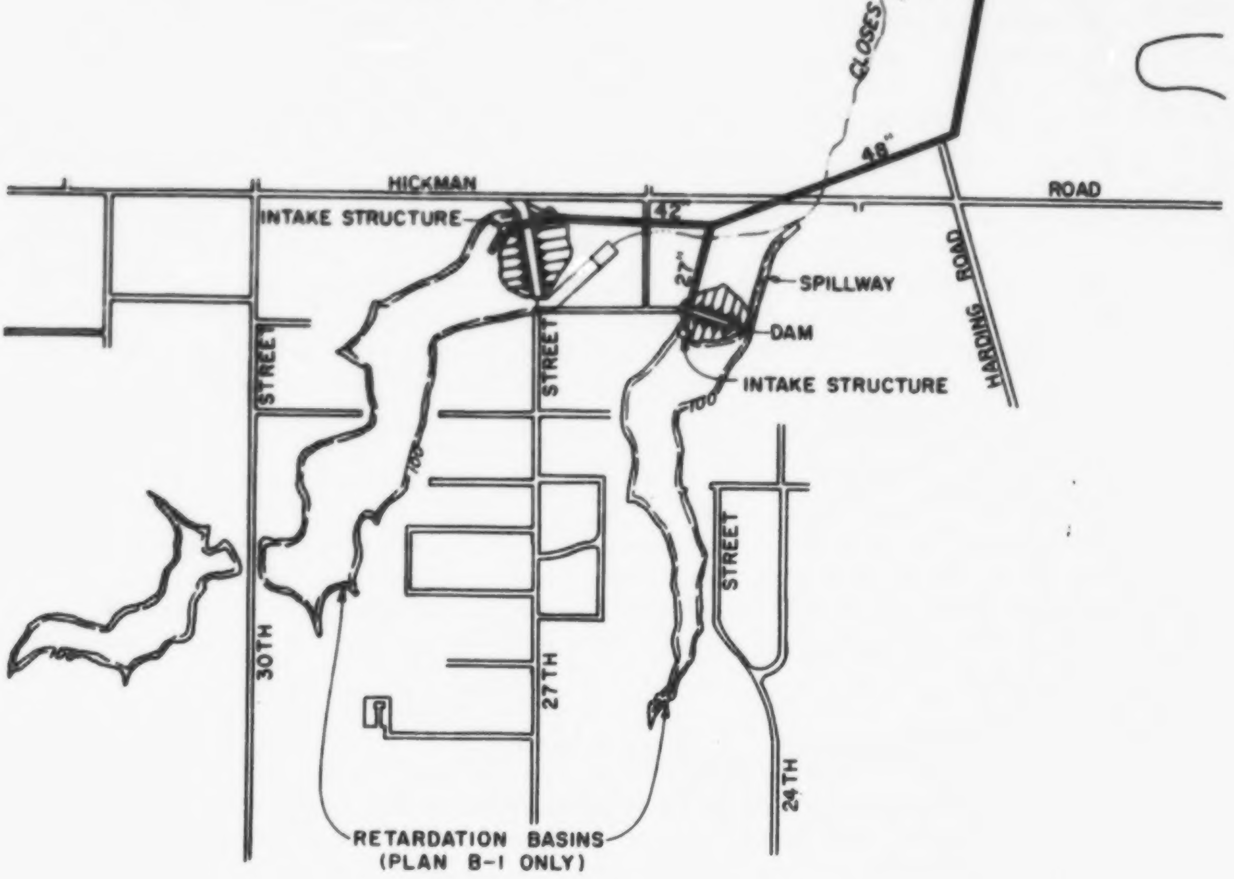
The estimated construction cost for the two retardation basins is $1,600,000, including $730,000 for land acquisition. Operation and maintenance includes periodic inspection, site maintenance, periodic cleaning, and operation of discharge facilities and are estimated to cost $7,600 annually.

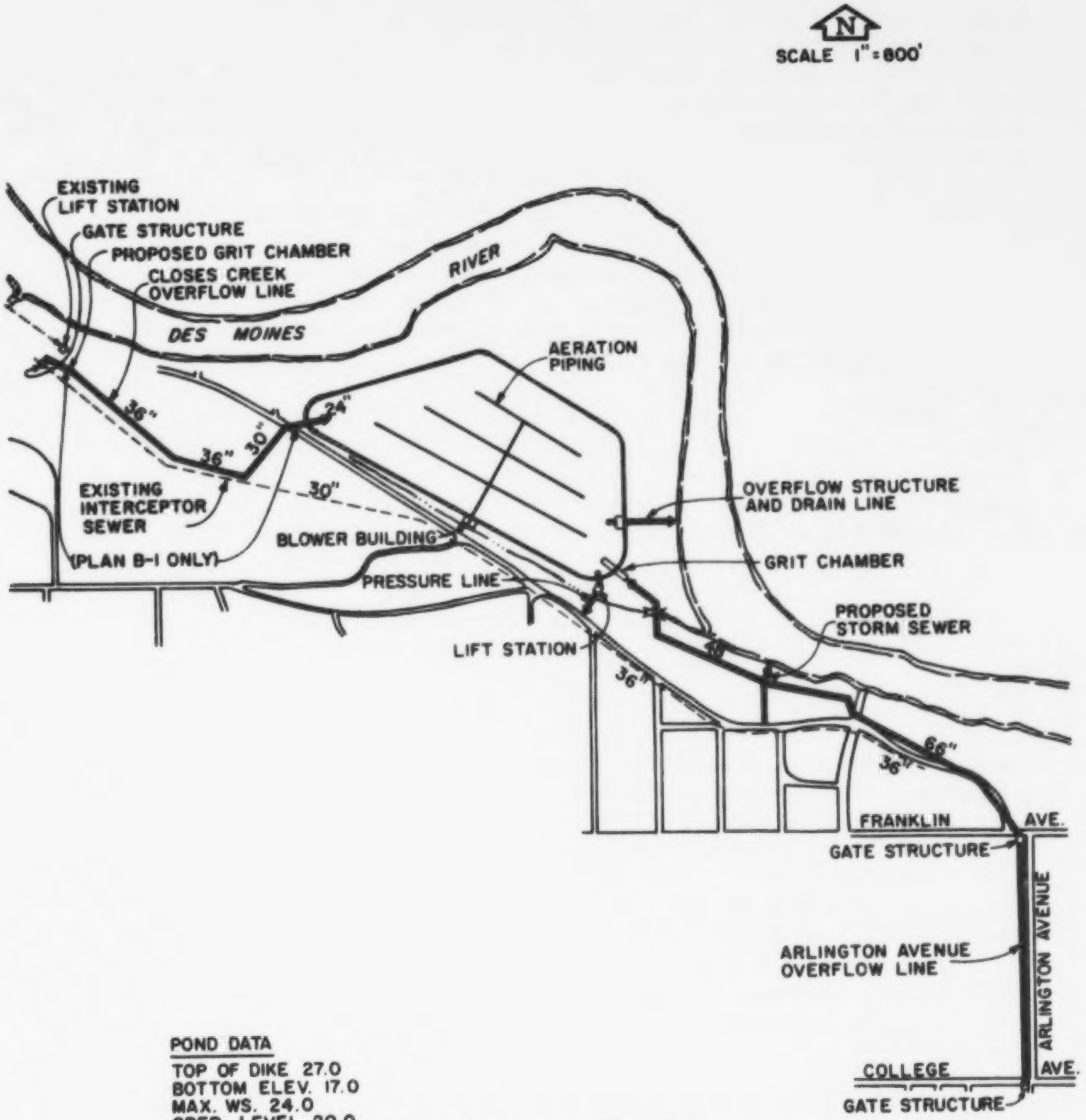
27th & Payne Separation. This is an area of only 8.85 acres at 27th Street and Payne Road, as shown on the location plat. It is in the Beaver Avenue System which is essentially a separate system pumped by the Prospect Road Lift Station. The cost of separation was minimal for this area since it is adjacent to a natural ravine. The construction cost is estimated to be $20,000. This project is included in both Plan B-1 and B-2.

7th & Franklin Separation. This is also a small area, 11.93 acres, located at 7th Street and Franklin Avenue. A small separate storm sewer located at this intersection discharges to the 36-inch West Side Interceptor one-half block east of the intersection. This separate storm sewer can be discharged to the Des Moines River at minimal cost. The estimated construction cost is $11,000. The project is included in both Plan B-1 and B-2.


14'
15'
11'
30'
℄ DIKE
48"
32'
CHUTE
PLAN
20.75
30.00
25.00
24.75
24.50
5'
28.00 DIKE
2'
20.00
70'
SECTION
STILLING BASIN
32' x 32'

TYPICAL GRIT CHAMBER
SCALE 1" = 30'


CLOSES
CREEK
48"
HICKMAN
ROAD
INTAKE STRUCTURE
42"
27"
SPILLWAY
DAM
INTAKE STRUCTURE
HARDING ROAD
30TH STREET
27TH STREET
24TH STREET
100
100
100
RETARDATION BASINS
(PLAN B-1 ONLY)


N
SCALE 1" = 800'
EXISTING LIFT STATION
GATE STRUCTURE
PROPOSED GRIT CHAMBER
CLOSES CREEK OVERFLOW LINE
RIVER
DES MOINES
AERATION PIPING
36"
36"
30"
24"
EXISTING INTERCEPTOR SEWER
30"
(PLAN B-1 ONLY)
BLOWER BUILDING
OVERFLOW STRUCTURE AND DRAIN LINE
GRIT CHAMBER
PRESSURE LINE
PROPOSED STORM SEWER
LIFT STATION
36"
48"
66"
36"
FRANKLIN
AVE.
GATE STRUCTURE
ARLINGTON AVENUE
ARLINGTON AVENUE OVERFLOW LINE
COLLEGE
AVE.
GATE STRUCTURE

POND DATA

TOP OF DIKE 27.0
BOTTOM ELEV. 17.0
MAX. WS. 24.0
OPER. LEVEL 20.0
OPER. STORAGE 99 AC. FT. for B-1, 71 AC. FT. for B-2
TOTAL STORAGE 168 AC. FT. for B-1, 121 AC. FT. for B-2

CLOSES CREEK SYSTEM

FIGURE 51

Prospect Road Impoundment. A holding pond would be constructed in the Crocker Woods area north of Prospect Road. The purpose of this pond is to store all combined sewage flows in excess of the interceptor capacity from Arlington Street and above, and to reduce flows in the West Side Interceptor sufficiently to prevent overflows from the interceptor above Grand Avenue. The areas contributing to the pond are designated on the Location Plat and the proposed facilities are shown in Figure 51.

For Plan B-1, combined flows would be picked up and carried by gravity sewers to the lagoon from the following points: On the 30-inch Closes Creek Sewer near the existing lift station, the Franklin Street Sewer at Arlington Street, and the College Avenue Sewer at Arlington Street. Sewers from the latter two points would be combined at Franklin Street enroute to the lagoon.

Grit chambers would be provided in each line before entering the lagoon. They consist of concrete basins with a transverse grit-storage channel as shown in Figure 51. Cleaning of the basins would be on a routine basis by a mobile crane with a clamshell bucket. The criteria for the grit chamber design is as follows:

Removal Efficiency 80% of 35 mesh grit.
Grit Storage - 20 CF/MG from 6" design storm.
Flow-through-velocity - 1.0 fps at maximum flow.

The operating capacity of the pond is 99 acre-feet, which is sufficient to contain the combined flows resulting from a 6-inch, 24-hour volumetric rainstorm. In addition, a minimum water depth of 3 feet would be maintained to prevent an odor nuisance from solids that may settle out and to maintain biological growth for treatment if required.

A lift station is provided to return stored sewage to the 36-inch interceptor sewer during low flow periods when the existing sewer system can handle it. An overflow and drawoff structure is also provided to discharge treated effluent to the river in the event excessive flows prohibit pumping the pond contents back to the system. With the modifications to the downstream system this should be a very infrequent occasion.

Because of the high BOD load to the pond, a diffused air system is provided to maintain aerobic conditions during storage. Aeration capacity is designed to satisfy the BOD requirement of the 6-inch design rain, but would not maintain solids in suspension. Diffusion piping is located in 3-foot deep, V-cut channels to provide adequate depth at low water levels and increase efficiency for peak demands.

If excessive flows prevent pumping the stored sewage back to the system, the contents of the pond would be stored for up to 7 days, then released at a controlled rate to the stream. The operating storage of the pond is designed to contain a 6-inch rainfall, whereas the average annual 24-hour maximum storm is a 2.7-inch rainfall. Very infrequently would a 7-day accumulation of rainfall equal 6 inches. Even if a 6-inch rainfall occurred when the pond was full, the pond would provide almost 2 days detention under aeration.

Criteria for design of the holding pond is as follows:

Store 6" design storm	99 Acre-feet, depth = 4.0 feet
Total Pond Volume	168 Acre-feet, total depth = 7.0 feet
BOD: 6" Design Storm-	65 mg/l, 16,870 lbs.
2.72" Annual Max. Storm-	75 mg/l, 11,600 lbs.

Assume 30% of BOD_5 exerted in first 24 hours of storage.

Blowers - 3 Each, firm capacity for 6" Design Storm,

5000 cfm plus standby

Effluent Lift Station - Firm Capacity of 3200 gpm

Pump down Operating Pool in 7 days.

Because this facility is located adjacent to a heavily used recreation area, the site would be fenced, suitable landscaping would be provided, and the site must be maintained in an attractive condition.

The estimated construction cost for the Prospect Road Impoundment is as follows:

(1)	Holding pond, complete, including land acquisition ($120,000), aeration system, structures, fencing and landscaping	$400,000
(2)	Line from Closes Creek bypass including grit removal	$ 65,000
(3)	Lines from Arlington and College Ave. Systems including grit removal	$412,000
(4)	Sewage Lift Station and Pressure Line	$ 75,000
	Total Estimated Construction Cost	$952,000

The estimated annual operation and maintenance cost including power, labor, equipment and supplies is $22, 500.

Closes Creek Separation With Impoundment (Plan B-2). The Closes Creek Watershed contains approximately 107 acres drained by combined sewers, which is everything west of Harding Road that is shown on the location plat as draining to the Prospect Road Impoundment. The sanitary drainage area is 1673 acres and the present population is estimated to be 12, 300.

The alternate presented in Plan B-2 is separation of the 107 acres served by combined sewers in the Closes Creek watershed. This is considered because of the large domestic load compared to the combined sewer area. The two retardation basins on Closes Creek would no longer be needed to treat combined sewer overflows. Separation of this area also would greatly reduce the flow and BOD to the Prospect Road Impoundment, thus reducing the volume and aeration requirements.

The areas to be separated include system numbers 9, 10, 11, 12 and 13 in the separation study area (Section IX) and approximately 10 additional acres outside of this area. The estimated construction cost of this separation is $511, 000. The Prospect Road Impoundment would be reduced in operating volume from 99 acre-feet to 71 acre-feet. The total volume of the reduced pond is 121 acre-feet. The depth remains the same. The Closes Creek overflow line and grit chamber would not be required, but the Arlington Avenue overflow line and grit removal unit would remain the same.

Because of the reduced flow and BOD load to the pond, float-type surface aerators are used in the reduced pond in place of a diffused air system. These will provide sufficient dissolved oxygen to maintain aerobic conditions during storage of the 6-inch design storm, but would not maintain all suspended solids in suspension.

During winter months, the float-type aerators would be removed from the pond due to freezing and the bypass gates on Arlington Avenue would be maintained closed to take all storm runoff to the system downstream. The volume of snow-melt or early spring and late fall storm runoff would not exceed the downstream capacity of the West Side Interceptor.

The effluent lift station remains as before and the overflow structure is included to provide the capability of overflowing to the river after a period of treatment, if required.

Criteria for design of the holding pond is as follows:

Store 6" Design Storm - 71 Acre-feet, Operation, depth = 4.0 feet

Total Pond Volume - 121 Acre-feet, Total depth = 7.0 feet

BOD: 6" Design Storm - 55 mg/l, 10,520 lbs.

2.72" Annual Max. Storm - 59 mg/l, 6,120 lbs.

Assume 30% of BOD_5 exerted in first 24 hours.

Aeration - 7 aerators for O_2 distribution @ 24 lbs. O_2 /hour each = 4200 lbs. O_2 per day.

Effluent Lift Station - Firm capacity of 3200 gpm, pump down operating pool in 5 days.

The estimated construction cost for this separation and the reduced holding pond is as follows:'

(1)	Holding Pond for 71 Ac-ft., complete, including land acquisition ($100,000), aeration system, structures, fencing and landscaping	$295,000
(2)	Line from Arlington Ave. Systems - including grit removal	$412,000
(3)	Sewage Lift Station and pressure line	$ 75,000
	Total estimated construction cost	$782,000

The estimated annual operation and maintenance cost including power, labor, equipment and supplies is $17,000.

West Side Interceptor

The area of concern is that area drained by combined sewers west of the Des Moines River and north of the Raccoon River which is indicated on the Location Plat as flowing to the Case Lake Treatment Complex. Also included is the area to be separated lying north of the Freeway along the Des Moines River. The service area includes the central business district west of the river. Land use is heavily commercial with medium and high density residential. The present population is approximately 25,000. An additional 25,000, served by the separate Northwest Outfall above the Closes Creek Basin, is also tributary to this system. The area drained by combined sewers is 1028 acres.

Separation of Freeway System (Plans B-1 and B-2). This area is located between the Freeway and University Avenue and between 2nd Avenue and 14th Street. It is served by separate sanitary sewers which are intercepted by a storm drainage system installed during construction of the Freeway. Flow is west to Keosauqua, and ultimately into a large system known as the Birds Run. The Birds Run Storm Sewer is an 8-foot diameter, or equivalent section, which discharges to the West Side Storm Box between Grand and Locust Streets. By virtue of this separation, combined sewage overflows to the Birds Run will be eliminated and this storm sewer will be discharged directly to the Des Moines River. This separation project is included in both Plan B-1 and B-2.

Separation would be accomplished by constructing a portion of Sanitary System F included in Plan A. The portion to be constructed would include lines between Manholes F13 to F33 to F12, F12 to F36 to F41. The estimated construction cost is $136,000.

Enlargement of West Side Storm Box (Plan B-1). The diversion of storm flows from the Closes Creek and Arlington Avenue areas will leave sufficient capacity in the West Side Interceptor to carry all other combined flows above Grand Avenue. At Grand Avenue a new bypass is provided which will divert excess storm flows to the West Side Storm Box below the Birds Run outlet. The section of the existing storm box above Birds Run will be abandoned and Birds Run will discharge directly to the river. For Plan B-1, the existing 6-foot by 13-foot storm box from this point to the Scott Street Dam would be reconstructed to carry all excess combined flows from the 60-inch West Side Interceptor. Figure 55 illustrates layout of the proposed improvements. An aerial view of the Des Moines River, looking north from the Scott Street Dam to the Central Business District, is shown in Figure 52.

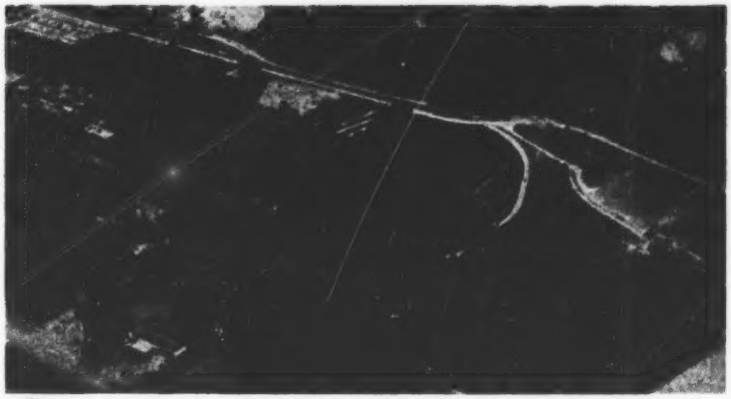
The new storm box will be 9 feet by 15 feet and will continue beyond the Scott Street Dam to the south bank of the river. The line would be a box section under the river bed and would terminate at a large grit removal chamber on the west bank of the river between the Chicago Great Western R. R. bridge and the Scott Street Dam. In addition to the Grand Avenue bypass, new connections to the storm box are provided at Walnut, Court, and Elm Streets.

The estimated construction cost of the work under Plan B-1 is as follows:

(1)	Reconstruction of the storm interceptor box	$920,000
(2)	Grand Avenue connection to storm box	36,000
(3)	Walnut Street connection to storm box	15,000

CASE LAKE
IMPOUNDMENT
AREA

DOWNTOWN DES MOINES -
LOCATION OF OVERFLOW INTERCEPTION
AND PUMPING FACILITIES, PLANS B-1 & B-2



E. 18TH STREET OVERFLOW IMPOUNDMENT

PROPOSED OVERFLOW POLLUTION
ABATEMENT FACILITIES

FIGURE 52

(4)	Court Street connection to storm box	$ 11,000
(5)	Elm Street connection to storm box	10,000
(6)	Extension of West Side Storm Box to Grit Chamber	480,000
	Total Estimated Construction Cost	$1,472,000

West Side Storm Box Improvements Without Enlargement (Plan B-2). As an alternate to expanding the West Side Storm Box, Plan B-2 calls for using the storm box at its present capacity and permitting a small volume of combined sewage to discharge to the river during high intensity storms. As shown in Table 17, overflows would occur at Walnut, Court and Elm Streets. For the 6-inch design storm the total discharge to the river from these three overflows amounts to 7.80 acre-feet, and for the 2.72-inch rain 3.54 acre-feet is discharged.

The existing box would be extended to the proposed grit chamber as in Plan B-1 and the Birds Run outlet modification would be constructed. Also, the new connections to the storm box would be made.

The estimated construction cost of the work to be accomplished under B-2 is as follows:

(1)	Grand Avenue connection to storm box	$ 36,000
(2)	Walnut Street connection to storm box	15,000
(3)	Court Street connection to storm box	11,000
(4)	Elm Street connection to storm box	10,000
(5)	Extension of West Side Storm Box to grit chamber	304,000
	Total Estimated Construction Cost	$376,000

Ingersoll Run System

Ingersoll Run Separation (Plan B-1). The Ingersoll Run System is a large combined sewer system west of the central business district and north of the Raccoon River. It is designated in the Location Plat as being the area to be separated under Plan B-1. This combined sewer system serves an area of 929 acres and has a population of approximately 10,000 people. Land use is medium and high density residential and commercial. The Ingersoll Run overflow discharges to the Raccoon River immediately east of the Fluer Drive Bridge.

A very limited area is available between this outlet and the Raccoon River, a portion of which is developed as an industrial area. Due to the limited area available, Plan B-1 is to separate this system to the extent necessary to eliminate combined sewage overflows. This would be accomplished by construction of Storm Sewer System Nos. 16, 19, 21 and 23, Sanitary Sewer Systems Nos. A, B, C and D, and a 215 acre portion of Storm Sewer Systems 24, 25 and 27. All the above systems are shown in the separation study area in Plan A. Including existing separate storm sewer systems, the area comprises 749 acres.

The estimated construction cost for this separation is $3,790,000.

Ingersoll Run Diversion (Plan B-2). The existing combined sewer systems in the Ingersoll Run Watershed would remain as they now exist under Plan B-2. A very simple diversion near the outlet of the Ingersoll Run overflow sewer would divert a portion of the overflow to the Southwest Outfall which runs past the outlet. The Southwest Outfall has reserve capacity from this point to the Main Outfall, based on estimated 2020 flows, to carry an additional 95 cfs. A gate structure would be provided on the diversion to limit flows to that amount. This additional flow in the Southwest Outfall would be bypassed to the overflow collection system at Scott Street for subsequent treatment at the Case Lake Treatment Complex.

Based on rainfall-runoff relationships developed in Section VII, the diversion of 95 cfs from this watershed would intercept rainfalls up to 0.33 inches per hour intensity. From the studies of rainfall intensities, it is estimated that this intensity would be exceeded by approximately 43 percent of the average annual March through November rainfall and 38 percent of the average annual rainfall. Also, this intensity is exceeded only 0.3 percent of the clock hours on an average annual basis, or approximately 26 hours annually.

The estimated construction cost for the diversion structure is $25,000.

East Side System

This area lies adjacent to the Des Moines River from Scott Street north to nearly the City limits. It contains an area of 2,240 acres, of which 544 are served by combined sewers. The present population is approximately 16,400. Land use is medium and high density residential and commercial.

The watershed is served by a 48-inch interceptor along the river from the Main Outfall at Scott Street to near University Avenue. A lift station located near Birdland Park discharges to a 36-inch gravity line on Pennsylvania Avenue which carries the pumped flow from the north to the 48-inch East Side Interceptor. The combined sewer area above the Birdland Station is 295 acres. Peak wet weather flows exceed the capacity of the pump station and the 36-inch gravity line. An overflow weir located at the pump station permits overflowing to the river.

East Side Overflow Sewer (Plan B-1). To eliminate the overflow at Birdland Station, a 48-inch gravity line from that point to Grand Avenue is proposed in Plan B-1. Because of a high bluff along the east bank of the river from Birdland Station to University Avenue, both tunneling and laying the pipe in encasement in the river were investigated. Tunneling was chosen because it was considerably cheaper than the river route. The tunnel section, approximately 3500 feet, is 54-inch diameter due to minimum economical tunnel size.

At Grand Avenue, the new 48-inch line is connected to the existing 48-inch East Side Interceptor, as shown in Figure 55. A new bypass to the East Side Storm Box is provided for excess storm flow from the Birdland area as well as for that contributed from the Grand Avenue combined sewer. New overflows are also provided at Walnut and Court Streets.

The existing East Side Storm Box has the capacity to carry all overflow from this system. At the termination of the existing storm box at the Scott Street Dam, a box section under the river bed will carry the storm flow to the large grit chamber on the west bank of the river.

The estimated construction cost of the items described above is as follows:

(1)	East Side Overflow Sewer from Birdland Station to Grand Avenue	$ 770,000
(2)	Grand Avenue junction box	15,000
(3)	Walnut Street connection to storm box	52,000
(4)	Court Street connection to storm box	46,000
(5)	Continuation of East Side storm box to grit chamber	775,000
	Total Estimated Construction Cost	$1,658,000

Birdland Park Retardation Basin (Plan B-2). As discussed previously, a new outfall to serve the growth north of Des Moines should be located on the east side of the Des Moines River at some time in the future. If the East Side overflow sewer from Birdland Station is to be constructed, the City of Des Moines should give consideration to increasing the size of this line to handle future flows from the north and making it a part of the northeast outfall system.

With this in mind, a retardation basin in the Birdland Park area was investigated under Plan B-2 as a means of deferring or eliminating the need for construction of a separate overflow sewer. This facility, located in the upper end of the Birdland Park Lake, is shown in Figure 53.

The basin would have an operating capacity of 23.2 acre-feet. This is sufficient to store the storm runoff in excess of the Birdland Station pumping capacity, based on a 6-inch design storm. In addition, a 3-foot low water depth would be maintained to prevent an odor nuisance from solids which would settle out.

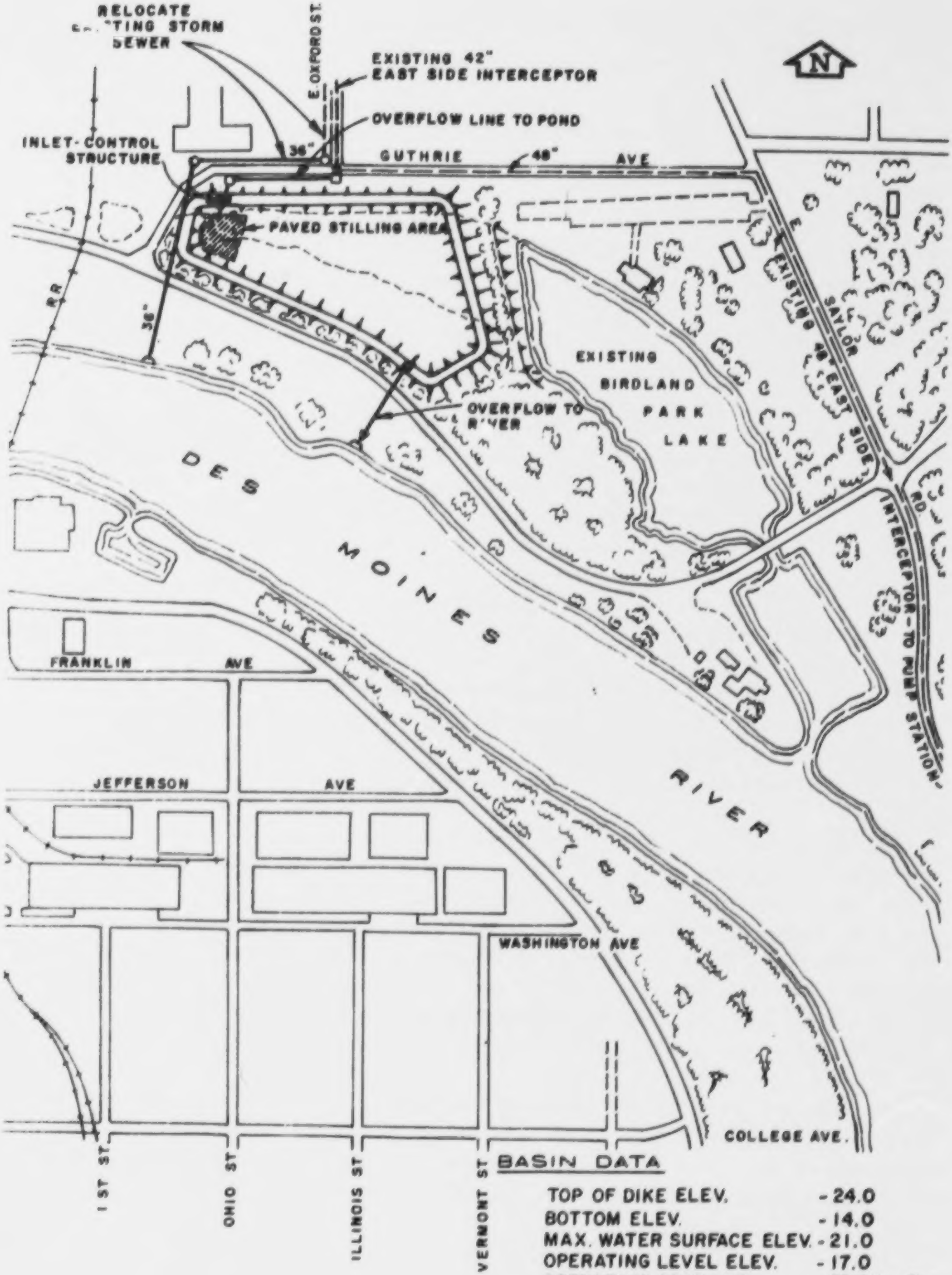
Flow to and from the basin would be by gravity. High water level in the basin would be at elevation 21.0, which is 6 inches above the crown of the 48-inch interceptor sewer, and the operating range would be 4.0 feet. An overflow structure is provided in the basin at the 21.5 level to prevent excessive surcharging in the interceptor and possible property damage. Inspection of manholes during the study indicated a surcharge considerably greater than this.

A diversion structure, located at Oxford Street and Guthrie Avenue, would be of the overflow-weir type with a flap gate for return of flow from the basin when the water level in the sewer recedes. The basin inlet structure would be provided with manually-operated, multiple-level return gates as well as a flap gate on the outlet. This would permit the basin to "float" on the level of the interceptor sewer or be operated as a holding pond, depending on whether the return gates were maintained open or closed.

Float-type surface aerators are provided to maintain aerobic conditions in the basin in the event excessive runoff is required to be stored. Because of the location of this facility in a heavily used park, extensive landscaping and screening would be required.

Criteria for design of the retardation basin is as follows:

Store 6" Design Storm - 23.2 Acre-feet, 4.0 feet deep


RELOCATE
EXISTING STORM
SEWER
E. OXFORD ST.
EXISTING 42"
EAST SIDE INTERCEPTOR
N
OVERFLOW LINE TO POND
INLET-CONTROL
STRUCTURE
36"
GUTHRIE
48"
AVE
PAVED STILLING AREA
RR
36"
EXISTING 48" EAST SIDE
SAYLOR
RD
EXISTING
BIRDLAND
PARK
LAKE
OVERFLOW TO
RIVER
DES MOINES RIVER
INTERCEPTOR - TO PUMP STATION
FRANKLIN AVE
JEFFERSON AVE
WASHINGTON AVE
COLLEGE AVE.
1 ST ST
OHIO ST
ILLINOIS ST
VERMONT ST

BASIN DATA

TOP OF DIKE ELEV.	- 24.0
BOTTOM ELEV.	- 14.0
MAX. WATER SURFACE ELEV.	- 21.0
OPERATING LEVEL ELEV.	- 17.0
OPERATING STORAGE	-23.2 Ac-Ft.
TOTAL STORAGE	-39.0 Ac-Ft.

BIRDLAND RETARDATION BASIN

FIGURE 53

Total Basin Volume - 39.0 Acre-feet, 7.0 feet deep

BOD: 6" Design Storm - 66 mg/l, 4170 lbs.

2.72" annual max. storm - 66 mg/l, 1650 lbs.

Assume 30% of BOD_5 exerted 1st day.

Aeration - 3 aerators @ 20 lbs. O_2/hour ea. = 1440 lbs. /day.

The estimated construction cost for the Birdland Retardation Basin is $224,000. Annual operation and maintenance costs, including power, labor, equipment and supplies are $10,000.

The estimated construction cost for the East Side System as proposed under Plan B-2 would be as follows:

(1)	Birdland Park Retardation Basin	$ 224,000
(2)	Grand Avenue Junction Box	15,000
(3)	Walnut Street connection to storm box	52,000
(4)	Court Street connection to storm box	46,000
(5)	Continuation of East Side storm box to grit chamber	775,000
	Total Estimated Construction Cost	$1,112,000

East 18th Street System (Plans B-1 and B-2)

This system, including the 20th Street system which is tributary to it, extends from 18th and Maury to the north City limits and lies adjacent to the East side Interceptor area. The area has heavy industrial development, as well as extensive commercial and residential areas. The present population is estimated to be 17,300, and the area includes approximately 3,000 acres. Approximately 150 acres, primarily residential areas, are served by combined sewers. These areas are located in the upper end of the East 18th street system as shown in the Location Plat, Figure 57.

To reduce the quantity of storm water mixing with the high strength industrial waste and to relieve flooding problems in the east 18th and Maury areas, excess combined flows will be divered to a lagoon adjoining the upper end of Dean Lake. An aerial view of the Dean Lake impoundment is seen in Figure 52. A diversion structure at East 18th and Court Avenue would route all combined sewage in excess of 11.5 cfs to the lagoon. The remainder would continue to the plant for treatment. The layout of the lagoon and piping is shown in Figure 54. Included is a 66-inch pipe from East 18th Street and Court Avenue to the lagoon and a 66-inch discharge line into the northern section of Dean Lake. The total capacity of the lagoon would be 54.60 acre feet with an operating capacity of 32.60 acre-feet between normal water level of 12.00 and maximum water level of 16.00. This will be more than adequate to store the runoff produced by a 6-inch design storm, which is estimated to be 18.7 acre-feet. Runoff would be stored for 7 to 10 days, then discharged at a controlled rate to Deans Lake.

A minimum of 3.0 feet of water would be maintained in the lagoon to prevent odor nuisances from solids that will settle out. Float-type surface aerators are provided to maintain aerobic conditions, but are required only after very heavy rainfalls. They would not be required during the winter months and would be removed to prevent damage from freezing.

Criteria for design of the Dean Lake Impoundment are as follows:

Store 6" Design Storm - 32.6 Ac-Ft., 4.0 feet deep

Total Pond Volume - 54.6 Ac-Ft., 7.0 feet deep

BOD: 6" Design Storm - 60 mg/l, 3200 lbs.

2.72" Annual Max. Storm - 60 mg/l, 764 lbs.

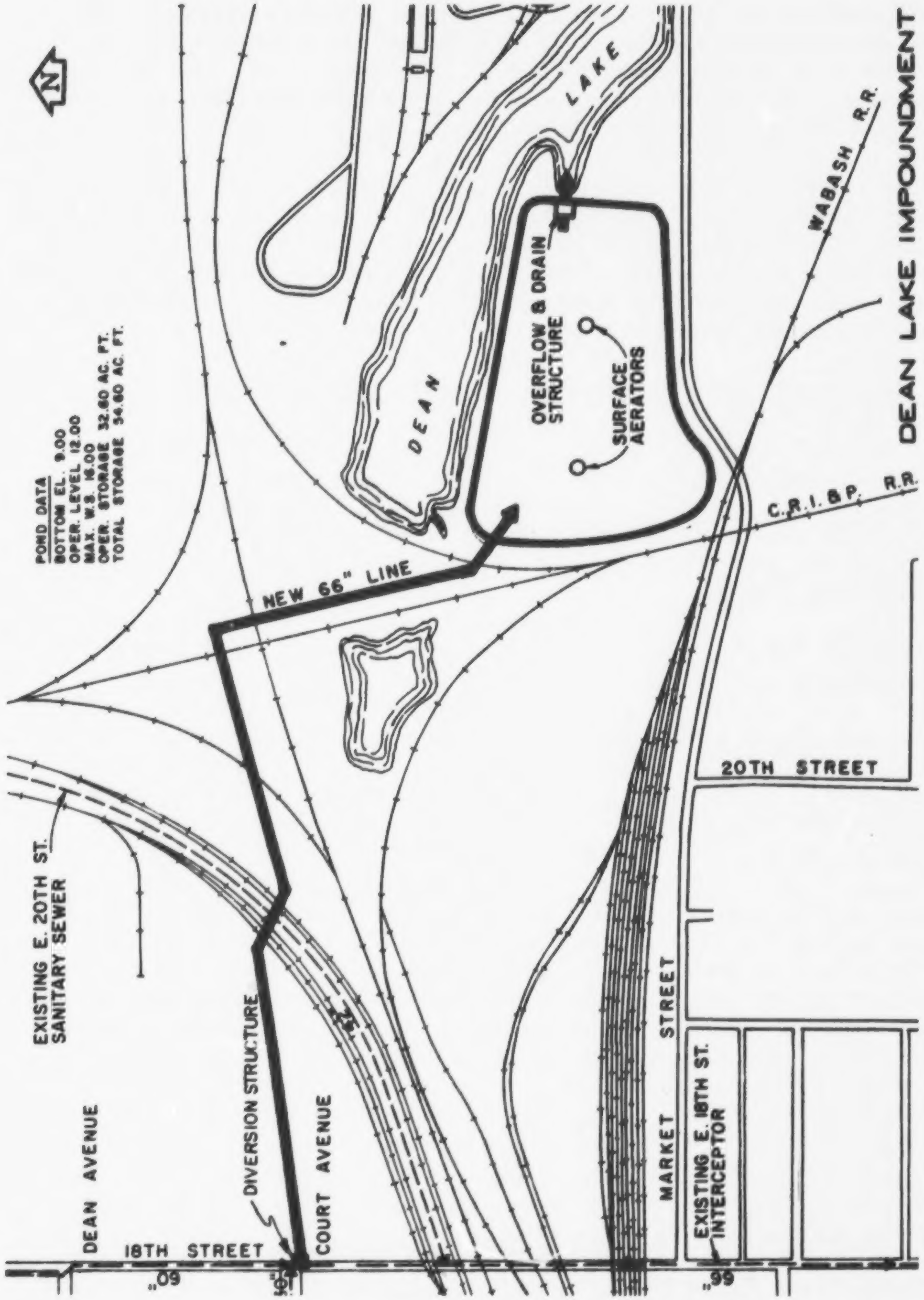
Assume 30% of BOD exerted first 24 hours storage.

Aeration - 2 aerators @ 18 lbs. O_2/hour ea. = 864 lbs/day

The estimated construction cost of the Dean Lake Impoundment and 66-inch Diversion Line is $371,000. Estimated operation and maintenance cost is $4,400 annually.

South Side Trunk (Plans B-1 and B-2).

The South Side Trunk drainage basin contains 2,061 acres and has a population at present of 15,300. Combined sewers serve 142 acres of this basin, shown on the Location Plat, Figure 57, as the combined sewer areas south of the Des Moines and Raccoon Rivers. The area is primarily residential with neighborhood commercial areas.


POND DATA
BOTTOM EL. 9.00
OPER. LEVEL 12.00
MAX. W.S. 15.00
OPER. STORAGE 32.60 AC. FT.
TOTAL STORAGE 34.60 AC. FT.
N
DEAN
LAKE
OVERFLOW & DRAIN STRUCTURE
SURFACE AERATORS
WABASH R.R.
C.R.I. & P. R.R.
NEW 66" LINE
20TH STREET
EXISTING E. 20TH ST. SANITARY SEWER
DIVERSION STRUCTURE
DEAN AVENUE
COURT AVENUE
MARKET STREET
EXISTING E. 18TH ST. INTERCEPTOR
18TH STREET
60"
66"
72"
66"

DEAN LAKE IMPOUNDMENT

FIGURE 54

All separate sanitary and combined flows from this area are tributary to the existing siphon structure and storm water pumping station at South East 9th and Jackson Streets. Combined flows from the low area adjacent to the Des Moines River are pumped to the siphon structure, where excess flow discharges to the river.

The existing lift station would remain in service and an additional lift station would be constructed to handle the overflow from the 9th Street line. The new lift station would discharge into the storm outfall line running to the Case Lake Treatment Facility. The estimated construction cost of the new lift station is $105, 000, and operation and maintenance costs are estimated to be $3, 100 annually.

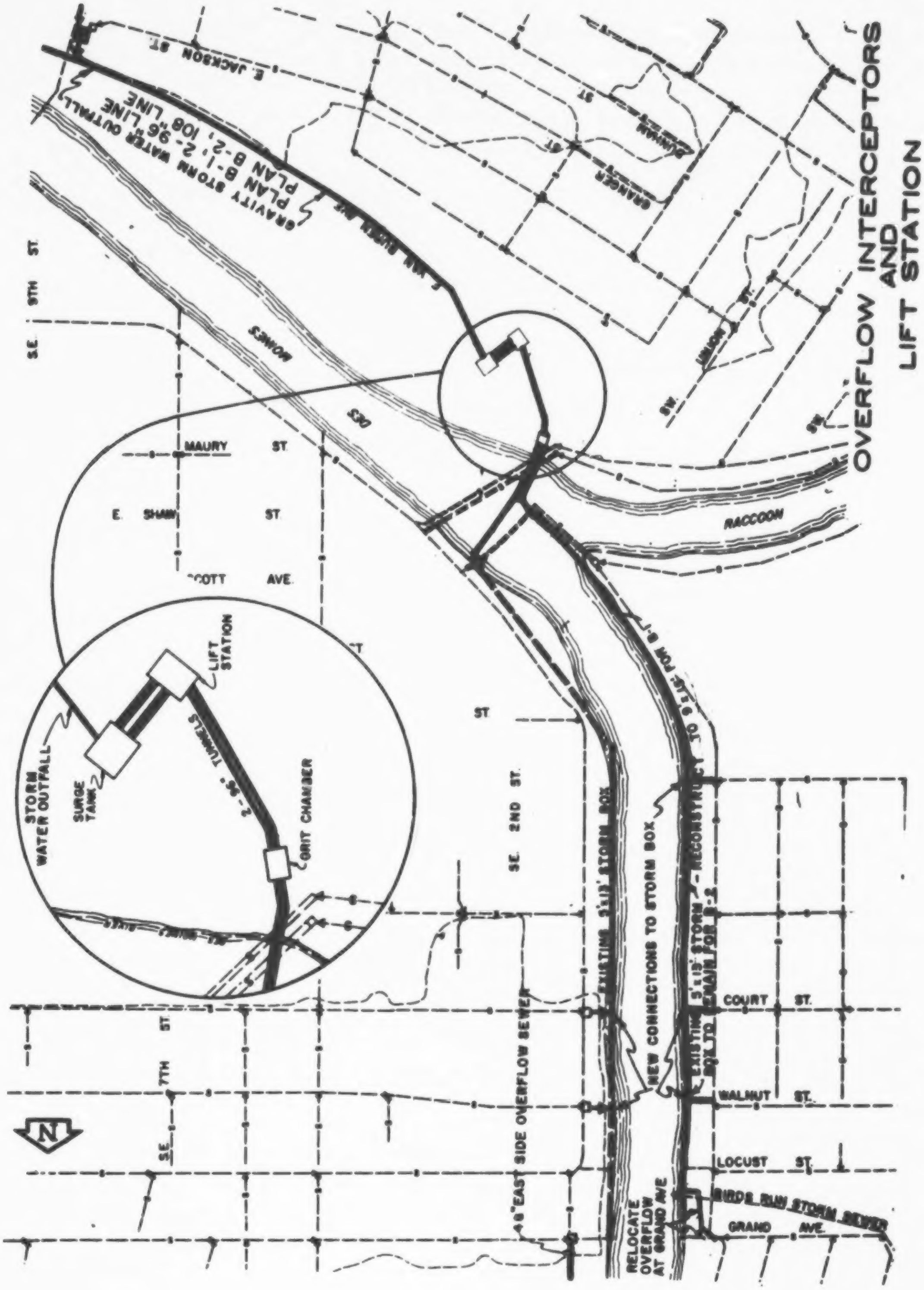
Scott Street Lift Station and Storm Outfall (Plans B-1 and B-2)

A large grit chamber would be constructed on the west bank of the Des Moines River between Scott Street and the Chicago Great Western Railroad Bridge. Storm flows from the East and West Side Storm Boxes would discharge to this chamber. Figures 52, 55 and the Location Plat, Figure 57, show the location of this chamber as well as the lift station and storm outfall.

The grit chamber would remove heavy grit and debris and would protect the lift station from damage. It would not remove fine grit since this is accomplished at the treatment facility. On the basis of 20 cubic feet of grit per million gallons, 3600 cubic feet of grit storage is provided. This estimate of the quantity of grit is conservative, but not unrealistic based upon experience with combined sewers in other locations (17). Cleaning of the basin would be on a routine basis by a mobile crane with a clamshell bucket.

A large lift station will be constructed just south of East Van Buren Avenue between SE 1st Street and SE 3rd Street. Under Plan B-1, this lift station would contain 2 - 100, 000 gpm and 2 - 50, 000 gpm pumps and would lift a maximum of 650 cfs of combined sewage approximately 47 feet into a surge tank. In Plan B-2, the lift station would handle peak flows of 500 cfs, using 2 - 80, 000 gpm and 2 - 40, 000 gpm pumps. The surge tank capacity would be sufficient to contain the peak flow for a period of 3 minutes. The grit chamber described above, would be connected to the lift station wet well by two 96-inch tunnels.

A gravity storm outfall would be constructed from the surge tank to a large lagoon complex in the Case Lake area, located on the south side of the river opposite the existing treatment plant. This gravity line would consist of two 96-inch diameter pipes under Plan B-1 and one 108-inch diameter line under Plan B-2. The lines would operate under


E. JACKSON ST.
GRAVITY STORM WATER OUTFALL
PLAN B-1, 2-96" LINE
PLAN B-2, 108" LINE
GRANGER ST.
SW. UNION ST.
SW.
S.E. 9TH ST.
DES MOINES
RACCOON
MAURY ST.
E. SHAW ST.
SCOTT AVE.
LIFT STATION
STORM WATER OUTFALL
SURGE TANK
2-96" TUNNELS
GRIT CHAMBER
ST.
SE. 2ND ST.
EXISTING 5'x15' STORM BOX
NEW CONNECTIONS TO STORM BOX
RECONSTRUCT TO 9'x15' FOR B-1
EXISTING 5'x15' STORM BOX TO REMAIN FOR B-2
COURT ST.
WALNUT ST.
LOCUST ST.
BIRDS RUN STORM SEWER
GRAND AVE.
RELOCATE OVERFLOW AT GRAND AVE
48" EAST SIDE OVERFLOW SEWER
SE. 7TH ST.
N

OVERFLOW INTERCEPTORS AND LIFT STATION

FIGURE 55

pressure during maximum pumping conditions. At East 9th Street, this line would pass above the existing gravity line which runs along East 9th Street and siphons the Des Moines River enroute to the Main Interceptor. The new storm overflow lift station for the South Side Trunk area discharges to the storm outfall at this point.

The estimated construction cost of the facilities is as follows:

	Item	Plan B-1	Plan B-2
(1)	Grit removal installation	$ 210,000	$ 210,000
(2)	Twin 96" tunnels from grit chamber to lift station	375,000	375,000
(3)	Lift Station	1,158,000	833,000
(4)	Pressure lines, lift station to surge tank	34,000	34,000
(5)	Surge tank	118,000	118,000
(6)	Storm outfall to Case Lake Treatment Complex		
	Plan B-1, 2 - 96" lines	2,569,000	
	Plan B-2, 108" line		2,167,000
Total Estimated Construction Cost		$4,464,000	$3,737,000

Estimated annual operation and maintenance costs are $48,000 for Plan B-1 and $44,000 for Plan B-2.

Case Lake Treatment Complex

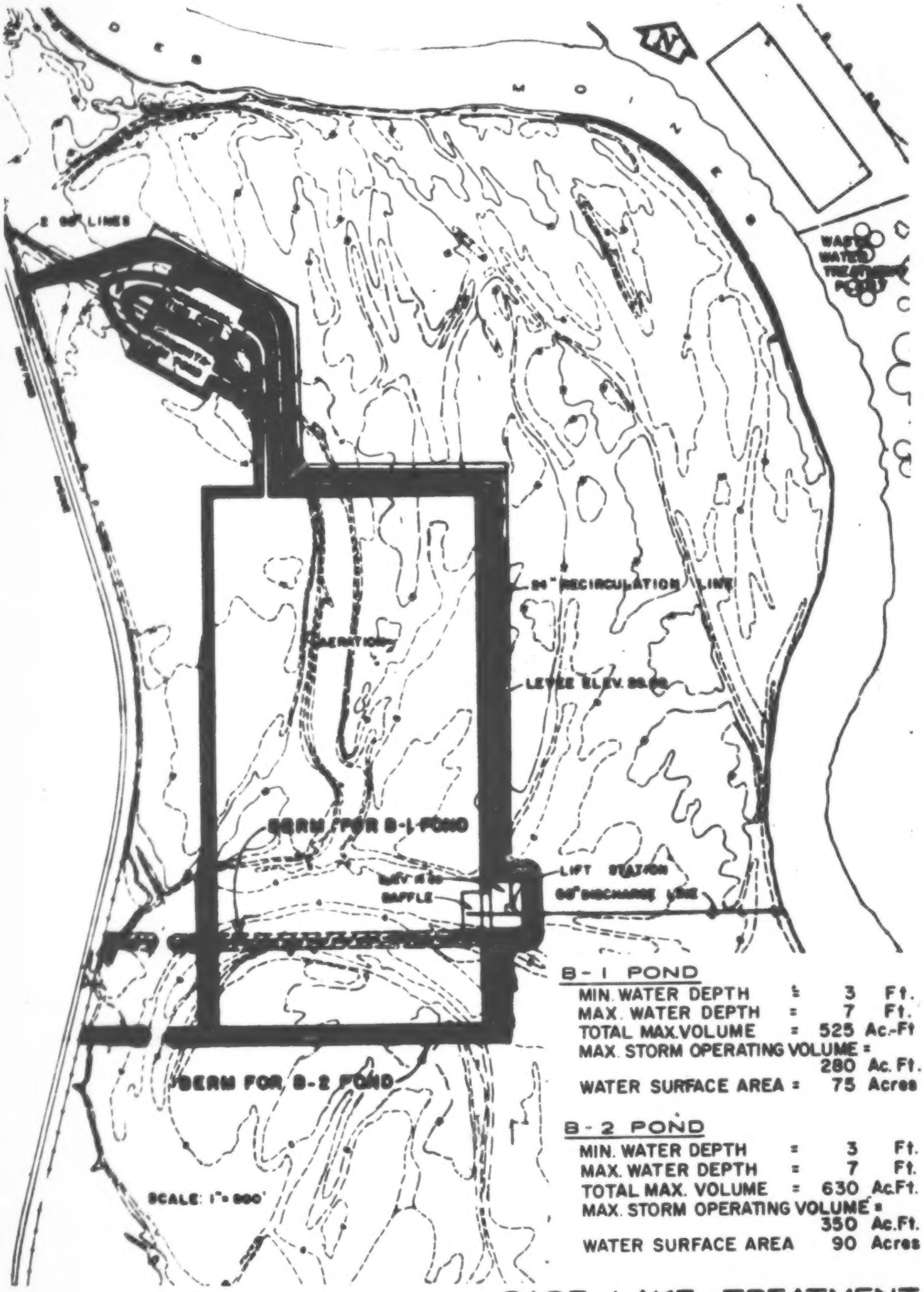
This treatment complex is designed to handle combined flow from the proposed storm water outfall. It is located in the Case Lake area directly across the river from the present treatment plant. The area is considered unusable for development by local planning authorities, partially because of location and partially because a portion of the area is within the maximum flood pool for Red Rock Reservoir. The infringement on the flood pool is minor, but has to be considered. The dikes are designed to withstand the maximum flood elevations. Since the complex would be handling flow that would otherwise add to the flood pool, it is possible the infringement can be discounted.

The general philosophy of the treatment complex was to provide a facility which could handle intermittent large flow from the storm water outfall. Storm water flow for the Plan B-1 design storm is 280 acre-feet/day, part of which may be received at a peak rate of 650 cfs. For Plan B-2, the design storm water volume to the Case Lake complex is 350 acre-feet/day, with peak flows of 510 cfs. To treat the above flows, two treatment schemes were considered. The difference in the two plans is the method of treating the overflow and the expected quality of the effluent. The basic plan, shown in Figure 56, includes sedimentation ponds for removal of grit and heavy solids followed by an aerated stabilization pond. This treatment scheme would be expected to provide a high degree of treatment. The alternate plan would be grit removal, primary clarification and chlorination. The effluent would not be as good as for the basic plan and serious operational problems would be expected due to the intermittent flows.

The Basic Plan - Lagoon Facilities. The component units of the basic plan are as follows:

(1) All storm water outfall would be discharged into two non-mechanical sedimentation ponds providing a minimum detention time of 15 minutes at 650 cfs peak flow. The ponds would be equipped with stop log gates at each end to permit alternate draining and cleaning. Cleaning would be accomplished with a front end loader. Raw BOD in the storm flow is expected to be about 60 mg/l. At 280 acre-feet of flow in Plan B-1, the raw BOD load would be 46,000 lbs/day. A 20 percent reduction is expected due to solids removal, leaving a total of 36,800 lbs/day going to the aerated stabilization pond. In Plan B-2, the raw BOD load would be 57,500 lbs/day and the effluent to the stabilization pond would contain 46,200 lbs/day of BOD.

(2) The effluent from the sedimentation ponds is discharged into an aerated stabilization pond designed to contain the design flow from the storm water outfall and hold this for a period of 6 days. If the pond were full when additional runoff occurred, detention would still be provided. Flow-through time for the runoff from the annual 2.72-inch rainfall would be 3 to 3.5 days, depending on the plan, if the pond were full. Noraml pond operation will be at a depth of 3 feet. The depth will increase to 7 feet during design storm conditions. A variable overflow will be required as will an effluent pump station for use during periods of high river stage. One of the effluent pumps will be used for recirculation to the sedimentation ponds to prevent septicity and increase mixing in the stabilization pond. Because of the magnitude of the shock BOD load from the storm water, aeration of the pond will be required. It is proposed to provide sufficient oxygen



D E S
M O I N E S
N
WASTE WATER TREATMENT PLANT
2 96" LINES
24" RECIRCULATION LINE
AERATION
LEVEE ELEV.
BERM FOR B-1 POND
LIFT STATION
96" DISCHARGE LINE
BAFFLE
BERM FOR B-2 POND
SCALE: 1" = 800'

B - 1 POND

MIN. WATER DEPTH = 3 Ft.
MAX. WATER DEPTH = 7 Ft.
TOTAL MAX. VOLUME = 525 Ac.-Ft
MAX. STORM OPERATING VOLUME = 280 Ac. Ft.
WATER SURFACE AREA = 75 Acres

B - 2 POND

MIN. WATER DEPTH = 3 Ft.
MAX. WATER DEPTH = 7 Ft.
TOTAL MAX. VOLUME = 630 Ac.Ft.
MAX. STORM OPERATING VOLUME = 350 Ac.Ft.
WATER SURFACE AREA 90 Acres

CASE LAKE TREATMENT COMPLEX

FIGURE 56

on a pound for pound basis to reduce the unit surface BOD load to 50 lbs/acre. Calculation of the air required considered 30 percent of the shock load BOD to occur in the first day. Based on the use of diffused aeration, the air requirement at 5 percent transfer efficiency is 6,700 cfm for Plan B-1 and 8,700 cfm for Plan B-2. It is estimated that the effluent BOD will be 20 to 25 mg/l, an effluent that should be satisfactory for present and anticipated future river conditions.

(3) An effluent pump station will be required for use during periods of high river stage. The proposed station will have one 30,000 gpm pump and two 7,500 gpm pumps. With this capacity, the storm water storage can be discharged in 72 hours. One of the 7,500 gpm pumps would be used for recirculation to the sedimentation ponds.

(4) In addition to construction of treatment units, some landscaping and drainage work will be required. An existing drainage channel will have to be routed around the pond system and through a levee which will be required for flood protection. (See Figure 56).

The estimated construction cost for the basic plan is:

		B-1	B-2
(1)	Sedimentation ponds	$ 120,400	$ 98,000
(2)	Aerated stabilization pond, including discharge line, Lift Station, recirculation line and area drainage	1,068,000	1,220,000
(3)	Aeration equipment	72,600	87,000
(4)	Blower building and equipment	73,000	83,000
Total Estimated Construction Cost		$1,334,000	$1,488,000

The estimated annual operation and maintenance cost, including power, labor, equipment and supplies is $24,000 for Plan B-1 and $28,000 for Plan B-2.

The Alternate Plan - Mechanical Plant. An alternate mechanical primary treatment scheme was also investigated. The plant would provide screening and grit removal, primary sedimentation, chlorination of the effluent and sludge storage ponds. It soon became apparent that the plan not only does not provide a comparable degree of treatment, but also was considerably more expensive, $5,700,000, as compared to the Lagoon System. A mechanical plant would provide serious operational problems due to the high intermittent flows. This would be particularly true during freezing weather when the units would have to be drained to prevent freezing. For obvious reasons, the plant was not considered feasible or satisfactory and so was not given further attention.

Summary of Construction Costs

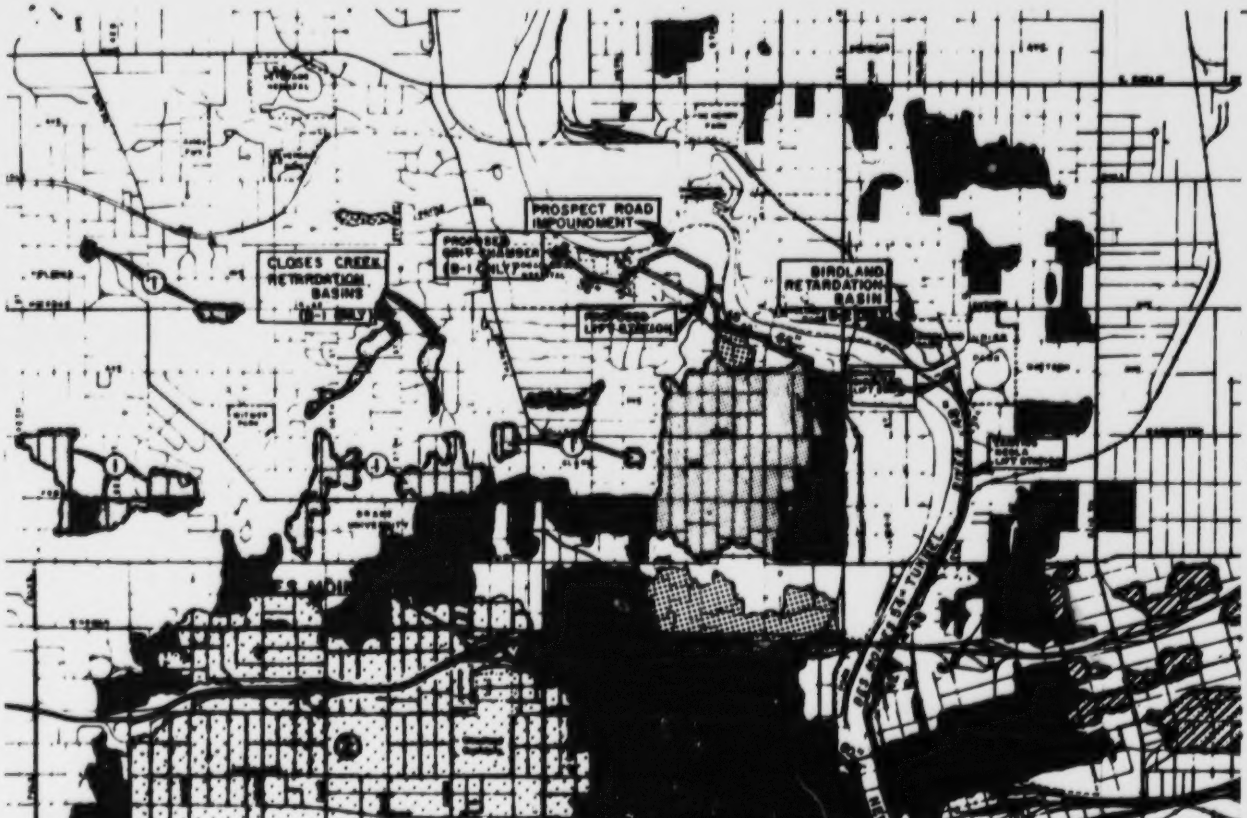
Summarized herein are the proposed improvements for each system described previously as they pertain to the two overall schemes for elimination or reduction of combined sewer overflows. Cost summaries are listed by system.

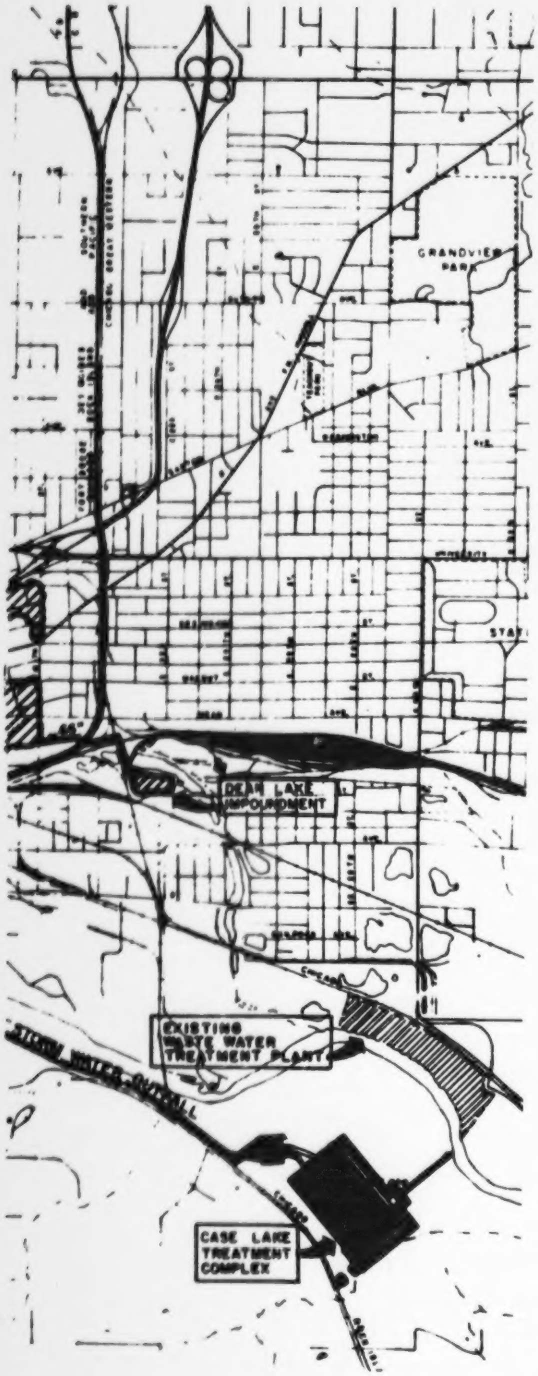
Plan B-1

1.	Closes Creek System	
	Closes Creek Retardation Basins	$ 1,600,000
	27th and Payne Separation	20,000
	7th and Franklin Separation	11,000
	Prospect Road Impoundment	952,000
2.	West Side Interceptor	
	Separation of Freeway System	136,000
	West Side Storm Box Enlargement	1,472,000
3.	Ingersoll Run Separation	3,790,000
4.	East Side System, including Overflow Sewer and East Side Storm Box Improvements	1,658,000
5.	East 18th Street System	371,000
6.	South Side Trunk	105,000
7.	Scott Street Lift Station and Storm Outfall	4,464,000
8.	Case Lake Treatment Complex	1,334,000
	Total Estimated Construction Cost-Plan B-1	$15,913,000

Plan B-2

1.	Closes Creek	
	Closes Creek Separation	$ 511,000
	27th and Payne Separation	20,000
	7th and Franklin Separation	11,000
	Reduced Prospect Road Impoundment	782,000
2.	West Side Interceptor	
	Separation of Freeway System	136,000
	West Side Storm Box as is and reduction in cost of extension to grit chamber	376,000
3.	Ingersoll Run Diversion	25,000
4.	East Side System	1,112,000
5.	East 18th Street System	371,000
6.	South Side Trunk	105,000
7.	Scott Street Lift Station and 108" Storm Outfall	3,737,000
8.	Case Lake Treatment Complex	1,488,000
	Total Estimated Construction Cost-Plan B-2	$8,674,000


PROSPECT ROAD IMPOUNDMENT
CLOSES CREEK RETARDATION BASINS
(B-1 ONLY)
PROPOSED GRIT CHAMBER (B-1 ONLY)
BIRDLAND RETARDATION BASIN
LIFT STATION
TUNNEL
RIVER
RACCOON
Grays Lake
PIONEER PARK


GRANDVIEW PARK
DEAN LAKE IMPOUNDMENT
EXISTING WASTE WATER TREATMENT PLANT
STORM WATER OUTFALL
CASE LAKE TREATMENT COMPLEX

N

LEGEND

┆ COMBINED SEWAGE DRAINAGE AREA TO PROSPECT ROAD IMPOUNDMENT

▨ COMBINED SEWAGE DRAINAGE AREA TO DEAN LAKE IMPOUNDMENT

■ COMBINED SEWAGE DRAINAGE AREA TO CASE LAKE TREATMENT COMPLEX

┆ DRAINAGE AREA TO BE SEPARATED UNDER PLAN B-1

▒ DRAINAGE AREA TO BE SEPARATED UNDER PLAN B-1 & B-2

① SEPARATED UNDER PLAN B-2

② TO CASE LAKE TREATMENT COMPLEX UNDER PLAN B-2 (SEPARATED UNDER PLAN B-1)

LOCATION PLAT
PLAN B-1 & B-2

FIGURE 57

STORM WATER POLLUTION ABATEMENT SYSTEMS

All sources of urban pollutant discharges need to be considered. A major source, whether considered a natural occurrence or man-made, is urban storm water discharges. Plan C includes the treatment of all storm water discharges in the Des Moines Metro Area.

The problems encountered in the collection and treatment of storm waters are similar to those of handling combined flows. The basic problem is to handle high volumes of flow at infrequent intervals of time. The facilities must have the capacity to effectively handle the flows from a given design storm runoff. It would be impractical to design a mechanical treatment facility to handle the flow rate from a 100-year storm. Such a facility may never be fully utilized and its cost effectiveness would be greatly reduced. A second problem is that land in urban areas is often highly developed. The methods of treatment selected have to be compatible with the development in the area or the flow will have to be conveyed to a suitable treatment site. Compounding this problem is the fact that storm water treatment facilities cannot be a centralized unit for an area such as Des Moines which encompasses approximately 49,000 acres. Treatment will be accomplished in smaller, more manageable segments. A third problem is having the capability financially to support a program of storm water treatment. Many communities such as Des Moines are involved in expansion programs for the treatment of sanitary wastewaters. It would be an extreme financial burden to increase the city's indebtedness and annual costs to the level required for treatment of storm waters.

Storm Water Discharge Characteristics

The data collected from storm water discharge sampling points is presented in Section VI, Table 7. The following observations are made following review of the data and comparison of the combined sewer overflows and storm water discharges:

(1) Storm water BOD values ranged from 40 to 70 mg/l.

(2) Storm water suspended solids values ranged from 400 to 600 mg/l. The concentrations of suspended solids was generally higher in the storm waters than in combined sewer overflows.

(3) Ammonia Nitrogen and Phosphate concentrations were lower in storm waters than in the combined overflows. This reflects the presence of raw sanitary wastes in the combined sewers.

(4) BOD and suspended solids concentrations are generally

highest in the initial flow of storm waters and combined sewer overflows. The concentrations characteristically decrease over the duration of the flow.

The data accumulated during the Des Moines Study compares favorably with other studies made of storm water discharges and combined sewer overflows. The comparative BOD concentrations of storm water and combined sewer overflow was the only possible exception noticed. Several other studies indicate BOD values in combined sewer overflow to be considerably greater than the concentrations in storm water discharges. In Atlanta, Georgia, BOD concentrations in combined sewer overflows were 50 percent greater than in storm water discharges (18). There was little difference in the BOD values as sampled in Des Moines. The concentration of BOD in these flows varies considerably depending on the characteristics of the locale, the sewer system, weather conditions and many other variables. For this reason, it is felt that the values as sampled in Des Moines should be used.

Methods of Storm Water Treatment

The treatment of storm water discharges is confined primarily to physical treatment methods. Biological systems are difficult to maintain in an environment where hydraulic flushing occurs at unpredictable intervals and where there are long periods of little or no flow. Some biological removal of organics can be expected if extended holding periods can be maintained in lagoon systems. Physical treatment such as short term detention, settling, and screening will remove high solids concentrations and thereby reduce the BOD by removal of settleable or screenable organic solids.

The schemes generally considered for treatment are long and short term detention ponds, grit removal units, settling clarifiers, coarse and fine screening, dissolved air flotation and chemical disinfection. Each of these methods has disadvantages. Lagooning requires large areas of isolated land and poses the problems of intense algae blooms, solids build up resulting from the lack of solids handling equipment, and the possibility of anaerobic conditions developing. The mechanical processes such as screening and conventional clarifiers involve equipment which will be a high maintenance item. Operational problems will be increased because the equipment will remain idle over two-thirds of each year. Operations using chemicals must be equipped with chemical feeders and meters large enough to handle very large ranges of flow. Again infrequent use and chemical storage will be a problem. To evaluate the costs of providing treatment to storm water discharges the following methods were reviewed.

(1) Grit removal units.

(2) Short term retardation basins.

(3) Clarifiers without mechanical solids removal.

(4) Screening, using coarse and fine units.

The volumes to be used for design were developed using the Rational Method. Runoff coefficients and intensities used were developed from the rainfall-runoff monitoring in Des Moines. These are shown in Figures 33, 34 and 35 of this report. Rainfall intensity-duration-frequency curves published by the U. S. Department of Commerce (12) and data furnished by the Des Moines Weather Bureau was also used. Table 19 gives a summary of the peak runoff flow rates and volumes considered for sizing the treatment units. The design basis for these systems was determined on the basis of storm intensity and frequency of return, considering the method of treatment to be evaluated.

Areas of Evaluation

Storm water treatment alternatives have been developed for four different areas within the study area. The areas were selected because sampling had been done on each and because they demonstrate the different types of areas which would require storm water treatment facilities. The areas are outlined on Figure 58 and described below:

1. Area I includes an area served by the Thompson Avenue Storm Sewer and additional areas to the northwest. The Thompson Avenue Storm sewer was sampled at Station S-1. The land area drained is about 1050 acres. The area is well developed and does not have large areas available for the use of lagoons or retention ponds.

2. Area II includes the area sampled at Station 0-2 on Closes Creek. The Closes Creek retardation basins, as described in this section, are designed to treat the total area runoff and are applicable to the evaluation in this section. Figure 51 provides a schematic of the basins. The two basins serve an area of 1387 acres.

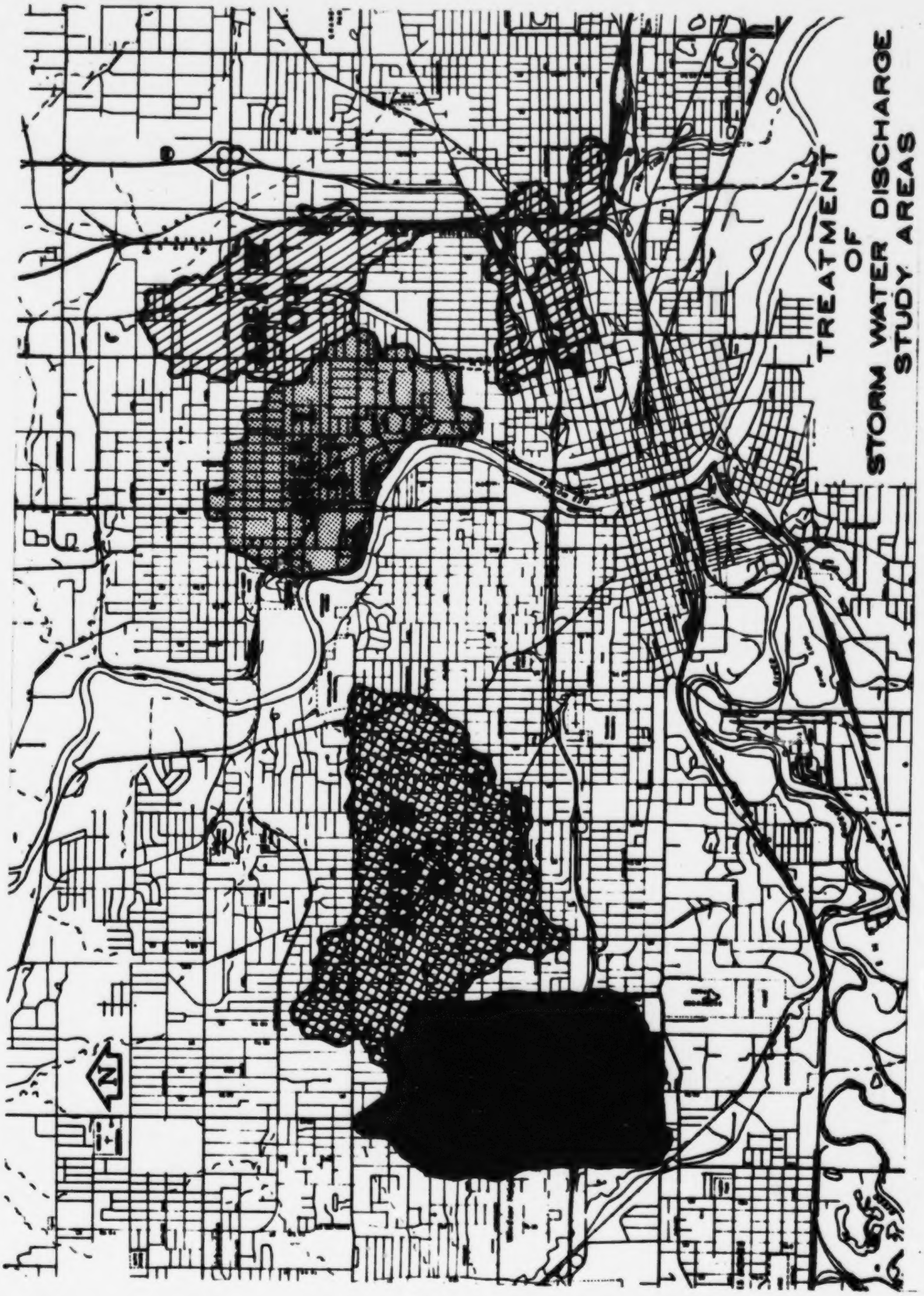
3. Area III includes the Cummin's Parkway Storm Sewer service area sampled at Station S-3 plus an additional area to the southeast. There are 1370 acres in the area. This area, like Area II, has a drainage way running through it which could be used for a short term retardation basin. The area is in a residential area with relatively high property values.

4. Area IV is the area served by the 20th Street Storm Sewer. The area was sampled at Station 0-11. This system drains toward an industrial area with some available open land areas. Area IV includes 1170 acres of land.

TABLE 19

STORM WATER DISCHARGES AT VARYING RETURN PERIODS
STORM WATER STUDY AREAS I - IV

DESIGN RAINFALL-FREQUENCY, INTENSITY, DEPTH	10 YEAR	1 YEAR	0.5 YEAR	0.2 YEAR
AREA I, A = 1050 ACRES, TIME OF CONCENTRATION Tc = 70 MIN.				
RAINFALL INTENSITY, I @ Tc = 70 MIN	2.0 INCHES/HR.	1.1 INCHES/HR.	0.9 INCHES/HR.	0.55 INCHES/HR.
COEFFIEIENT OF PEAK RUNOFF FLOW, C	0.50	0.43	0.42	0.36
PEAK FLOW RATE, Q = CIA	1050 cfs	496 cfs	398 cfs	208 cfs
24 HR. RAINFALL ACCUMULATION	4.75 INCHES	2.72 INCHES	2.25 INCHES	1.50 INCHES
VOLUMETRIC COEFFIEIENT OF RUNOFF, Cv	0.45	0.33	0.29	0.23
VOLUME OF RUNOFF	187 ac-ft	79 ac-ft	57 ac-ft	30 ac-ft
AREA II, A = 1387 ACRES, TIME OF CONCENTRATION Tc = 70 MIN.				
RAINFALL INTENSITY, I @ Tc = 70 MIN.	2.0 INCHES/HR.	1.1 INCHES/HR.	0.9 INCHES/HR.	0.55 INCHES/HR.
COEFFICIENT OF PEAK RUNOFF FLOW, C	0.50	0.43	0.42	0.36
PEAK FLOW RATE, Q = CIA	1387 cfs	656 cfs	525 cfs	275 cfs
24 HR. RAINFALL ACCUMULATION	4.75 INCHES	2.72 INCHES	2.25 INCHES	1.50 INCHES
VOLUMETRIC COEFFICIENT OF RUNOFF, Cv	0.46	0.36	0.33	0.27
VOLUME OF RUNOFF	253 ac-ft	113 ac-ft	86 ac-ft	47 ac-ft
AREA III, A = 1370 ACRES, TIME OF CONCENTRATION, Tc = 70 MIN.				
RAINFALL INTENSITY, I @ Tc = 70 MIN.	2.0 INCHES/HR.	1.1 INCHES/HR.	0.9 INCHES/HR.	0.55 INCHES/HR.
COEFFICIENT OF PEAK RUNOFF FLOW, C	0.50	0.43	0.42	0.36
PEAK FLOW RATE, Q = CIA	1370 cfs	649 cfs	518 cfs	272 cfs
24 HR. RAINFALL ACCUMULATION	4.75 INCHES	2.72 INCHES	2.25 INCHES	1.50 INCHES
VOLUMETRIC COEFFICIENT OF RUNOFF, Cv	0.45	0.33	0.29	0.23
VOLUME OF RUNOFF	244 ac-ft	103 ac-ft	75 ac-ft	39 ac-ft
AREA IV, A = 1170 ACRES, TIME OF CONCENTRATION Tc = 85 MIN.				
RAINFALL INTENSITY, I @ Tc = 85 MIN.	1.8 INCHES/HR.	0.85 INCHES/HR.	0.72 INCHES/HR.	0.50 INCHES/HR.
COEFFICIENT OF PEAK RUNOFF FLOW, C	0.53	0.44	0.42	0.38
PEAK FLOW RATE, Q = CIA	1115 cfs	437 cfs	354 cfs	222 cfs
24 HR. RAINFALL ACCUMULATION	4.75 INCHES	2.72 INCHES	2.25 INCHES	1.50 INCHES
VOLUMETRIC COEFFIEIENT OF RUNOFF, Cv	0.48	0.38	0.34	0.29
VOLUME OF RUNOFF	222 ac-ft	101 ac-ft	75 ac-ft	42 ac-ft



TREATMENT OF STORM WATER DISCHARGE STUDY AREAS

FIGURE 58

Treatment Schemes and Costs, Areas I, II, III, and IV

The following gives the design basis, the treatment method, system layouts, and the estimated construction and operating costs for storm water treatment for the four areas evaluated. Capital and operating costs are also given on a per acre basis, for ease of comparison.

1. **Area I** land area = 1050 acres, sampled at station S-1

Design Basis:

0.5-year return rainfall intensity = 0.9 inches/hour

Peak Flow rate = 400 cfs

Containment of one-year 2.72 inch storm = 80 ac-ft.

Treatment Method See Figure 59.

Alternate A includes a gravity collection system, grit removal units, a pump station, an 84-inch pressure sewer to the lagoon, and a 24 acre retention lagoon.

Alternate B includes a gravity collection system grit removal unit, a pump station, microscreens, and solids handling facilities.

Estimated Construction and Operating Costs

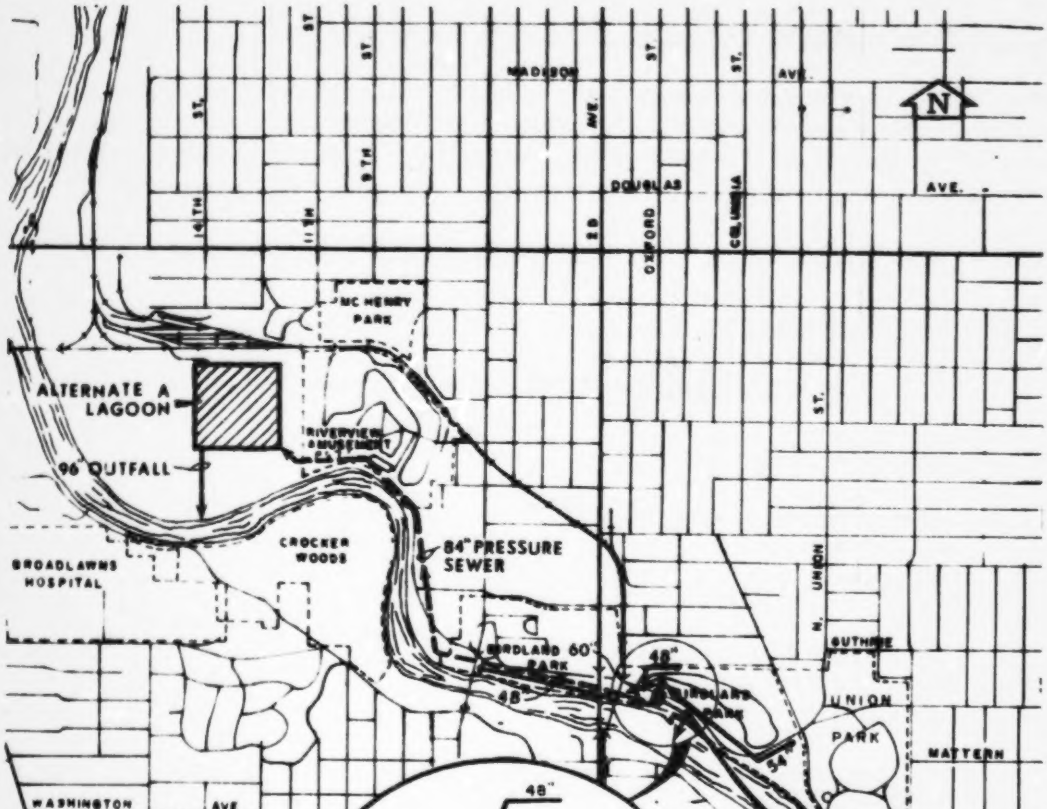
Gravity Collection System Alternate A and Alternate B

Collection Lines	$ 559,400
Non-mechanically cleaned grit chambers	350,000
450 cfs Pump Station	700,000
	$1,609,400

Alternate A: Lagoon

84" Pressure sewer to Lagoon	$ 567,000
24 Ac. Lagoon and 96" Outfall	680,000
Collection, Grit Chamber, Pump Station	1,609,400
Fees, Overhead, Contingency	573,600

Total Construction Cost = $3,430,000
Construction Cost per Acre = $3270/Acre
Annual Operations Cost = $9000/Year
Annual Operations Cost/acre = $8.55/Acre/Year


MADISON
AVE.
DOUGLAS
AVE.
14TH ST.
11TH
9TH ST.
2D AVE.
OXFORD ST.
COLUMBIA ST.
MC HENRY PARK
ALTERNATE A LAGOON
RIVERVIEW AMUSEMENT
96" OUTFALL
CROCKER WOODS
BROADLAWNS HOSPITAL
84" PRESSURE SEWER
BIRDLAND 60" PARK
48
48"
BIRDLAND PARK
N. UNION ST.
GUTHRIE
UNION PARK
MATTERN
54"
WASHINGTON AVE
COLLEGE
CLARK
ALTERNATE B SCREENING UNIT
48"
60"
COLLECTION STRUCTURE
54"
GRIT REMOVAL
84"
PUMP STATION
AVE.
GOOD PARK
ST.
DES MOINES RIVER
PK.
E 9TH
WALKER
CENTER
ST.
PARK
GRAND
State Capitol
N

STORM WATER DISCHARGE
TREATMENT LAYOUT
AREA I

FIGURE 59

Alternate B: Microscreens

Screens and Screening Building	$2,156,000
96" Outfall	60,000
Solids Thickeners and Handling	400,000
Collection, Grit Chamber, Pump Station	1,609,400
Fees, Overhead, Contingency	844,600
Total Construction Cost	$5,060,000

Construction Cost per Acre = $4820/Acre
Annual Operations Cost = $18,000/Year
Annual Operations Cost/Acre= $17.10/Acre/Year

2. Area II land area = 1387 acres, sampled at station 0-2

Design Basis

Spillway design for 100-year storm

For treatment-containment of 3-inch rainfall

Combined operating storage - 208 ac-ft.

Treatment Method - See Figure 51.

Retention Lagoons using existing drainage ways of Closes Creek.

Estimated Construction and Operating Costs. These estimates are taken from Section X and are itemized therein.

Total Construction Cost	$1,600,000
Construction Cost per Acre	$1150/Acre
Annual Operations Cost	$7600/Year
Annual Operations Cost/acre	$5.50/Acre/Year

3. Area III land area = 1370 acres, sampled at station S-3

Design Basis

Spillway designed for 100-year storm

0.5-year return rainfall intensity = 0.9 inches/hour

Annual Peak flow rate = 518 cfs

Containment of one-year 2.72-inch storm volume = 103 ac-ft.

Treatment Method. See Figure 60

Retention lagoons using existing drainage ways.

Estimated Construction and Operating Costs

Retention Lagoon	$ 866,400
Spillway	255,000
Site Work, Grit Stilling, Bridge	74,600
Overhead and Contingency	254,000
Total Construction Cost	$1,450,000

Construction Cost per Acre	$1060/Acre
Annual Operations Cost	$7000/Year
Annual Operations Cost/Acre	$5.10/Acre/Year

4. Area IV land area = 1170 acres, sampled at station 0-11

Design Basis

One-year return rainfall intensity = 0.85 inches/hour

Peak flow rate = 440 cfs

One-year 2.72-inch storm volume = 101 Ac-Ft.

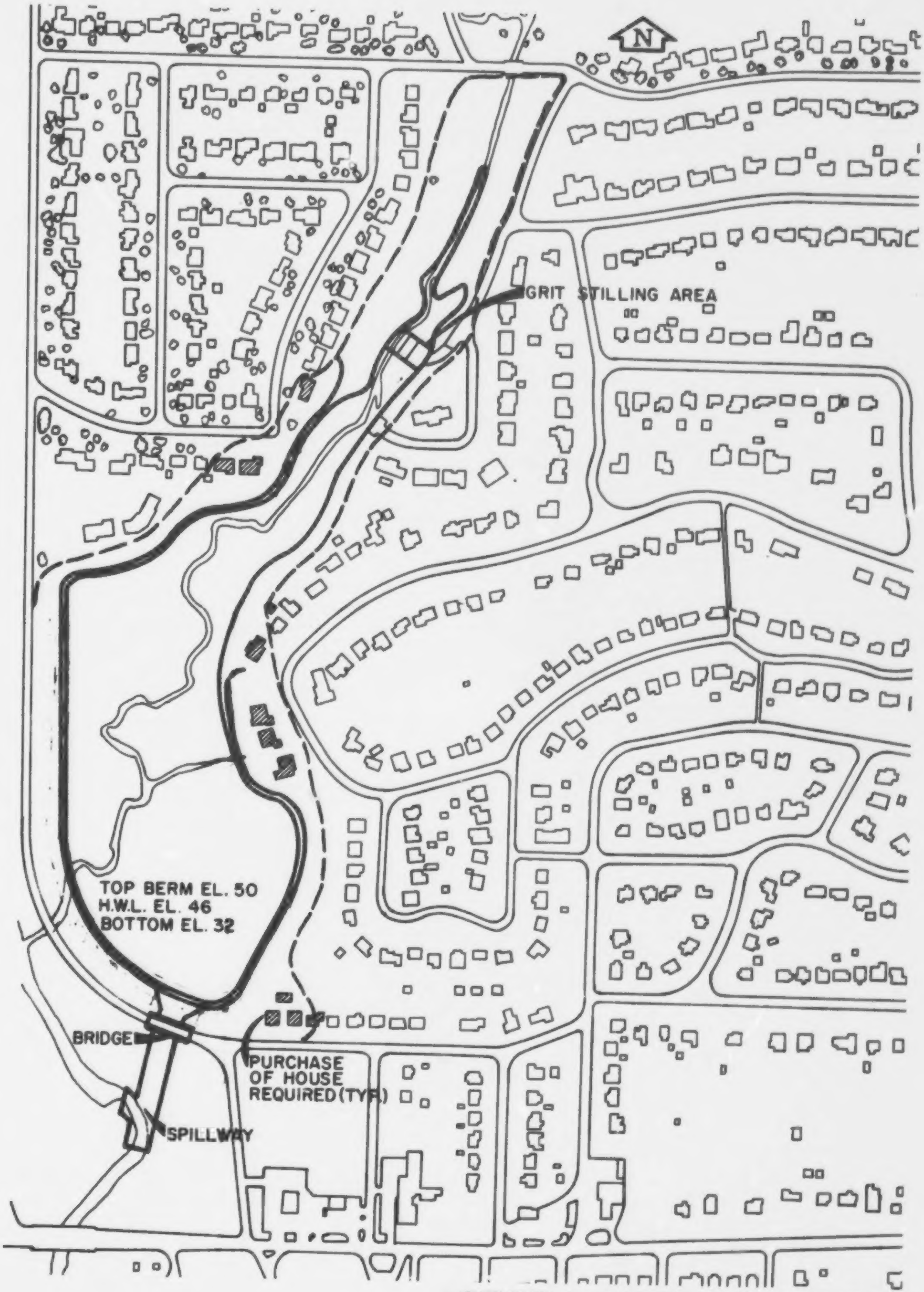
Treatment Method. See Figure 61. Grit chambers followed by non-mechanical rectangular clarifiers. Solids removal will be done periodically by clamshell or by front end loaders.

Estimated Construction and Operating Costs

Collection piping, site piping	$ 644,500
Land, Site work	200,000
Grit Chambers	326,000
Clarifiers, dewatering pumps	2,142,000
Overhead & Contingency	662,500
Total Construction Cost	$3,975,000

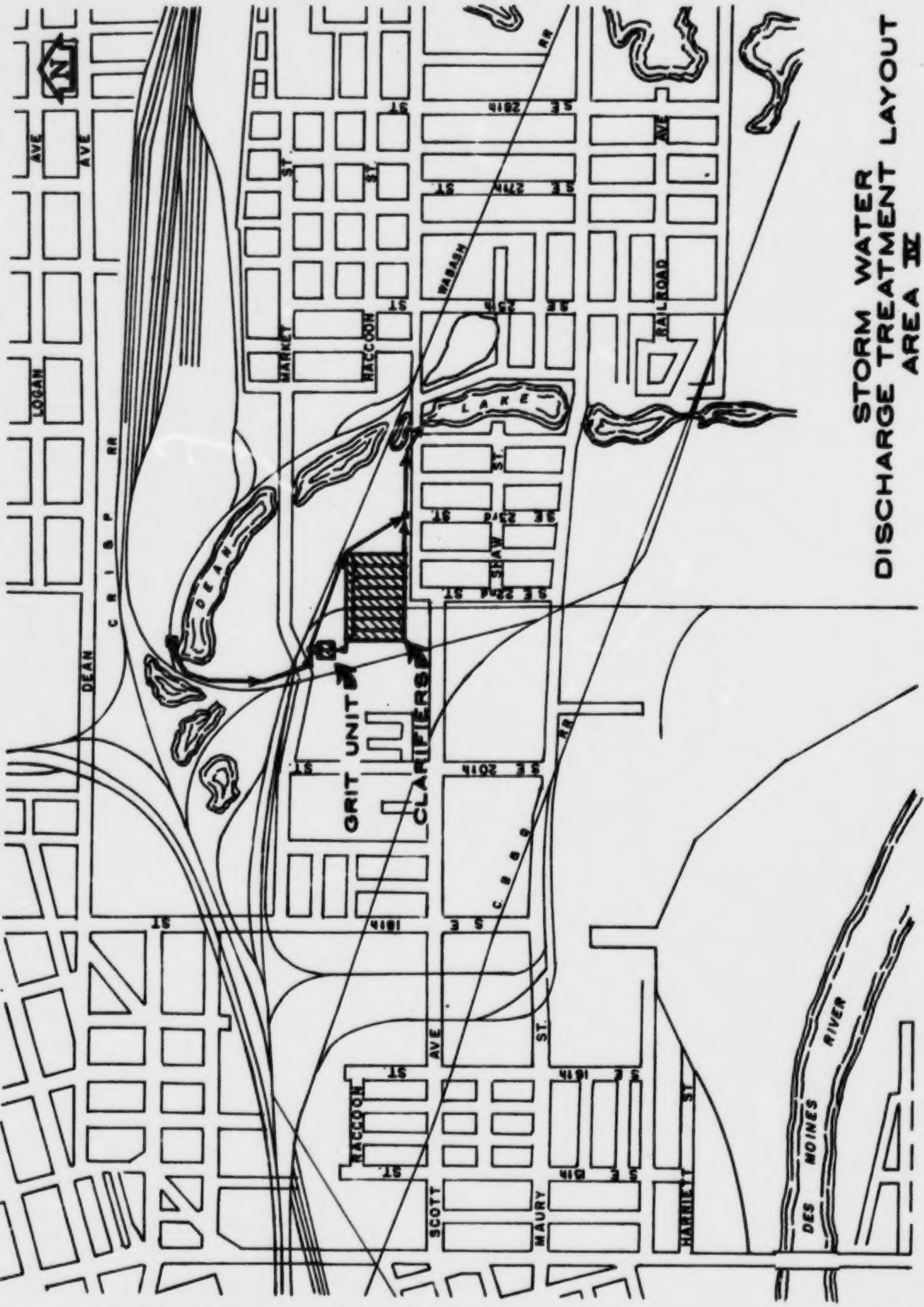
Construction Cost per Acre	$3390/Acre
Annual Operations Cost	$9500/Year
Annual Operations Cost/Acre	$8.10/Acre/Year

The costs of construction and operation are summarized in Table 20 along with the average values of the five evaluations.


N
GRIT STILLING AREA
TOP BERM EL. 50
H.W.L. EL. 46
BOTTOM EL. 32
BRIDGE
PURCHASE OF HOUSE REQUIRED (TYP.)
SPILLWAY

STORM WATER DISCHARGE
TREATMENT LAYOUT
AREA III

FIGURE 60


N
AVE
AVE
LOGAN
DEAN
C R I & P RR
MARKET ST.
RACCOON ST.
WABASH RR
S E 28th ST
S E 27th ST
S E 25th ST
S E 23rd ST
S E 22nd ST
S E 20th ST
S E 18th ST
S E 16th ST
S E 15th ST
AVE
RAILROAD
LAKE
ST.
SHAW
DEAN
GRIT UNIT
CLARIFIERS
C B & Q RR
RACCOON ST
SCOTT AVE
MAURY ST.
HARRIETT ST
DES MOINES RIVER

STORM WATER
DISCHARGE TREATMENT LAYOUT
AREA IV

FIGURE 61

TABLE 20

Costs, Storm Water Discharge Treatment

Area Evaluated	Treatment Method	Construction Cost/Acre	Operation Cost/ Acre/Year
Area I, Alt. A	Pump to Lagoons	$3,270	$ 8.55
Area I, Alt. B	Pump to Microscreens	4,820	17.10
Area II	2 Retardation Basins	1,150	5.50
Area III	Retardation Basin	1,060	5.10
Area IV	Non-mechanical clarifiers	3,390	8.10
Average	--	$2,738	$ 8.45

Projecting these per acre costs over the entire Des Moines Metro Area, the estimated costs are $134,700,000 for construction, and $415,700 per year for operation of the storm water treatment facilities.

Evaluation of Plans

Priorities need to be assigned to the abatement of the several types of discharges from the Des Moines Metro Area. The aim should be to spend the money so that the most benefit is derived. This is commonly referred to as cost effectiveness. An effort is made to show the effectiveness of each Plan and to discuss their benefits. Bear in mind that the alternative plans outlined present different concepts and provide different degrees of discharge abatement. Briefly, the plans presented are as follows:

Plan A, complete separation of the combined sewer system; treatment of only the separate sanitary wastes.

Plan B-1, treatment of all combined overflows, including some separation of combined sewers.

Plan B-2, treatment of most of the combined overflow, including a small amount of separation of combined sewers.

Plan C, treatment of all combined overflow and storm water discharges. Each plan includes the improvements and additions for the wastewater treatment plant as recommended by Vennstra and Kimm in Part B of the Central Iowa Regional Planning Commission report, (15).

Cost estimates have been provided earlier in this Section and are summarized in Section XIII. The costs used for this evaluation are the annual costs to build, finance and operate the facilities of each plan, including the costs of the wastewater treatment plant improvements and operation. The annual costs are taken from Table 24 of this report and from the Veenstra and Kimm report. These costs are totaled and summarized in Table 21.

TABLE 21

Metro Area Treatment Plans, Annual Cost, $/Year

	P&I*	O&M	Total
Proposed WWTP Facilities	$ 2,615,400	$1,600,000	$ 4,215,400
Plan A	4,472,500	1,400,000**	5,872,500
Plan B-1	4,214,500	1,709,600	5,924,100
Plan B-2	3,485,000	1,706,500	5,191,500
Plan C	17,719,100	2,125,300	19,844,400

*Amortized for 20 years at 6% interest

**WWTP O&M costs reduced to $1,400,000/year because of decreased operational costs with separated system.

Table 14 of this report lists the estimated present annual Metro area discharges. Each plan provides a different degree of treatment of the discharges. This fact gives some latitude in final decision making as to how much treatment to provide. From the loads in Table 14 the parameters least affected by the different Plans are the nitrates and ortho-phosphates. The schemes developed for the treatment of combined overflow and storm water discharges will not effectively remove these two constituents. This is true of most physical and biological treatment schemes. If nutrient treatment were required it would be most economical to remove phosphates chemically at the wastewater treatment plant. It is doubtful that even with significant reduction in these two parameters that the quality of the receiving waters would change noticeably. As stated previously in the evaluation of the river data, nuisance algal growths can occur with inorganic nitrogen levels of 0.3 mg/l and inorganic phosphorous exceeding 0.05 mg/l. The average values sampled at river station R-2 on the Des Moines River above the city exceeds these concentrations. If an algal problem were to develop, it could do so without the Metro Area nutrient loads because of the upstream rural contributions.

The BOD discharged from the Metro Area will vary with the Plan utilized. An estimate of BOD loads discharged by each plan is shown in Table 22. The Table was calculated using data from Tables 13 and 14 and the following criteria:

1. The average annual discharge of the proposed wastewater treatment facilities will meet the State discharge requirement of 7500 lbs/day of BOD.
2. Combined sewer overflow treatment facilities will discharge an effluent with 25 mg/l BOD.
3. Storm treatment facilities will provide on the average, a 50 percent BOD removal efficiency.

From the estimated BOD loads in Table 22 and the respective annual costs of each plan, justification of an extensive treatment program beyond that provided by the proposed new wastewater treatment plant facilities is difficult. Of the four plans, Plan B-2 appears to be the most favorable considering the results produced for the money spent.

The recreational use of surface waters in Des Moines makes the bacterial aspect of combined overflow treatment an important factor. The disinfection of these flows is accomplished in the treatment systems proposed by providing long detention times in the lagoons. The lagoons are generally sized to contain the volume of flow from a 6-inch storm. Nearly all of the overflow and storm water volume will be considerably less than this 6-inch storm.

The cost of providing chlorination of these storm flows would add considerably to the project costs. A 1970 study (19) which included disinfection costs, estimated capital costs of overflow treatment at $1000 per acre for conventional contact periods and $900 for a two minute contact period. These costs did not include land and engineering costs.

TABLE 22

Metro Area BOD Loads for Treatment Plans - Lbs/Year

Load	Present	With new WWTP	Plan A	Plan B-1	Plan B-2	Plan C
Effluent WWTP	6,259,000	2,737,500	2,737,500	2,737,500	2,737,500	2,737,500
"Wet" dry overflow	2,235,600	--	--	--	--	--
Combined flow treated @ WWTP	--	48,000	--	48,000	48,000	48,000
Untreated Combined overflow	174,500	174,500	--	--	2,500	--
Treated Combined Overflow	--	--	--	62,500	61,500	62,500
Untreated storm waters	2,668,000	2,668,000	2,899,700	2,668,000	2,668,000	--
Treated storm waters	--	---	--	--	--	1,334,000
Totals	11,385,100	5,628,000	5,637,200	5,516,000	5,517,500	4,182,000
Reduction from present	--	5,751,100	5,747,900	5,869,100	5,867,600	7,203,100

If disinfection was required, priority would have to be given to combined overflows. The combined overflow has the higher and potentially more hazardous bacteria levels. Data collected in Sacramento, California (20) on urban runoff and overflows showed that the mean combined overflow fecal coliform count was about one thousand times that of the storm water runoff. Obviously, the source of the combined overflow coliforms is more likely to be from domestic sewage than from storm water.

The operation of the overflow and runoff facilities can be an important factor in the containment of pollutants. Several studies, this one included, indicate that the first portion of the combined and storm flows carry the heaviest pollutional load. This occurs because of the flushing action during the initial runoff. Consequently, the beginning of a storm flow is important from the standpoint of retention. If bypass is to occur, the flow at the end of the storm should be that which is bypassed. Actual operating procedure would have to be based on sampling and operating experience. Controlled discharge to the stream during dry weather conditions should be based on effluent sampling. To provide capacity for subsequent runoffs, discharge should be accomplished as soon as effluent quality will permit.

The result of the evaluation is that the best return on the dollar would come from the development of the proposed wastewater treatment facilities. The costs of separation or treatment of overflows and runoff are quite high relative to the reduction in pollutional load achieved. The decreased load, beyond the improvement of the wastewater treatment plant, is not significant to the area water quality when compared to the upstream rural loadings. However, separation or treatment of combined sewer overflows can be used on a smaller scale to eliminate specific problems, to protect recreational waters, or to make system improvements concurrent with other reconstruction programs.

SECTION XI

UNUSUAL PROBLEMS ENCOUNTERED

This section describes some of the unusual problems encountered during execution of the project. The purpose for including this Section is two-fold: (1) awareness of the problems may be helpful in the development of future studies; and (2) identify items which can and should be resolved by the entities or agencies involved.

FIELD EQUIPMENT

Generally, once the field equipment had been received and installed, the performance thereof was satisfactory. The problem was in scheduling the initial selection, procurement, and delivery thereof. Although there are numerous equipment suppliers in this field, none of the items available completely satisfied all the specific requirements of this project. These particulars are discussed in Section V. Procurement of equipment took longer than was originally scheduled due to the specialized nature of many items. It took time to search out pieces of equipment which could be purchased separately and assembled at the project site or as in the case of the bubbler units, to prepare specifications for items to be fabricated by contract. Because many of the pieces were special order items, delivery was often delayed. For the same reason, items received were not always suitable for the purpose intended, causing additional delay for modifications or procurement of other equipment. The lesson to be learned is to schedule sufficient time and funds for equipment procurement and installation when developing project time and financial schedules.

A problem involving field equipment that must be solved is protection against vandalism. This was accomplished very successfully in this project by providing lock-down steel housing for all exposed equipment. For the bubblers, the housing was incorporated into the design and fabrication of the unit. For the float recorders and automatic samplers, 55 gallon steel drums were used. Slotted strap-iron flanges were welded to the base of the drum to permit the drums to be chained and padlocked to anchor bolts attached to a base. Although somewhat cumbersome during servicing of the units, this provided excellent protection against vandalism.

EXTENDED FLOODING

The problem which had the most profound effect on project operations was the extended period and reoccurrence of high river stage, first from spring runoff and then from upstream rainfall. The time interval extended

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from March to mid-August. During this period most of the major overflow points were either submerged or at least intermittently affected by high water. At several points, installation of monitoring and sampling equipment was physically impossible. Since the period involved is also the rainy season, the volume of data which might otherwise have been collected was materially reduced. The only apparent solution for this problem would be extension of the project through a second, hopefully dryer, year. Figures 62, 63 and 64 show photos of some of the problems caused by flooding.

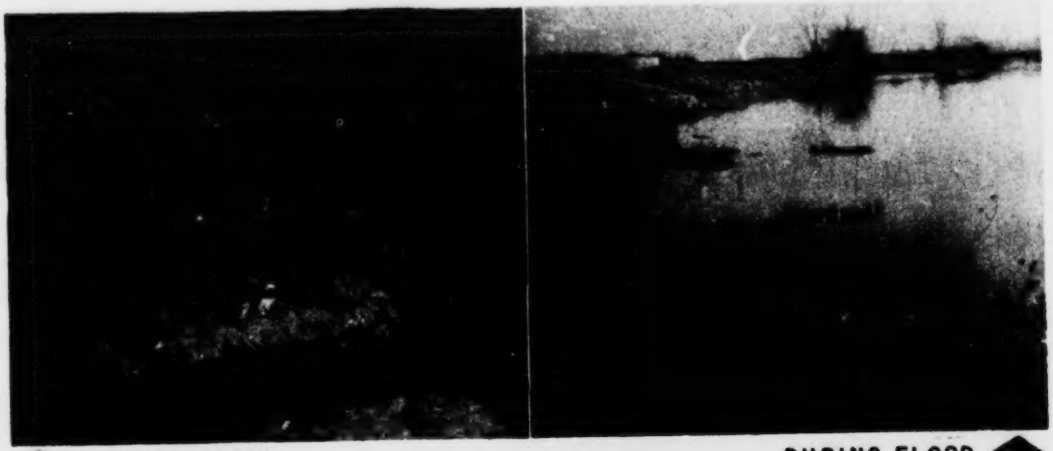
EXCESSIVE INFILTRATION

Excessive infiltration into the metropolitan area sanitary and combined sewage collection system was a second major problem. During the rainy period from May until mid-August, excessive infiltration caused almost continuous overflow at several of the key monitoring points. This problem was aggravated by the extended high river stage. To evaluate the magnitude of the infiltration problem, a network of flow measuring stations was established. Instantaneous flows were determined from current meter and depth observations. This data was then compared with the dry weather flows measured earlier in the project. The measuring station locations and flow data are shown in Figure 65. It is noted that sewage flows during the "wet" dry weather period reached 2 to 3 times the dry weather flows in some sections, and that the condition is not limited to any particular section of the city. Because of the general origin of flows, it is believed that building footing drains are a primary source of the problem. Sewer infiltration should vary with age and type of construction, however, high flows were measured in sewers serving relatively new areas. Although the infiltration flows were abnormally high during the study period, these flows have a major impact on the operation of existing collection and treatment facilities and must be given consideration in future design.

AVAILABLE RECORDS

The City of Des Moines has a system of sewer quarter-section maps. Because parts of the combined system are very old, accurate records are not available. In the past, limited personnel had prevented the staff from adequately maintaining some of these records. This problem is recognized by the City and work is being done whenever time is available to update these records.

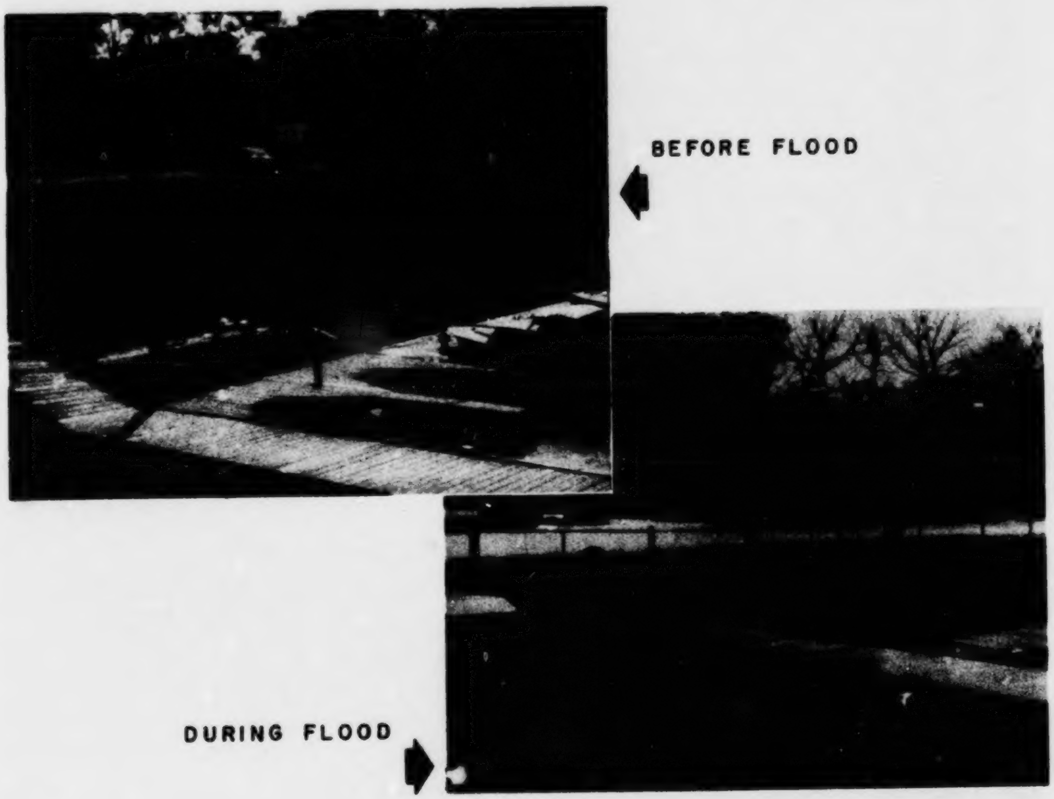
The lack of up-to-date records of storm sewers and inlet locations necessitated the expenditure of additional time to field check and update the information on the quarter section maps. Generally this work was accomplished during non-runoff periods and did not affect collection of



BEFORE FLOOD

DURING FLOOD

"RESCUING" BUBBLER
AT
WASTE WATER TREATMENT PLANT BYPASS



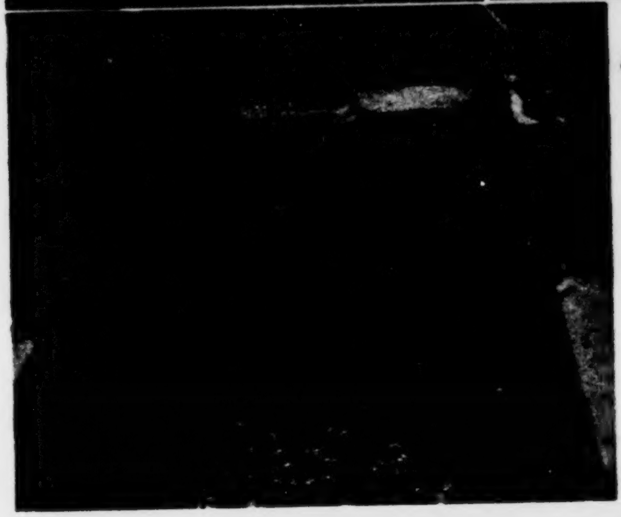
BEFORE FLOOD

DURING FLOOD

AT STATION S-1

NORMAL & FLOOD CONDITIONS AT OUTLETS

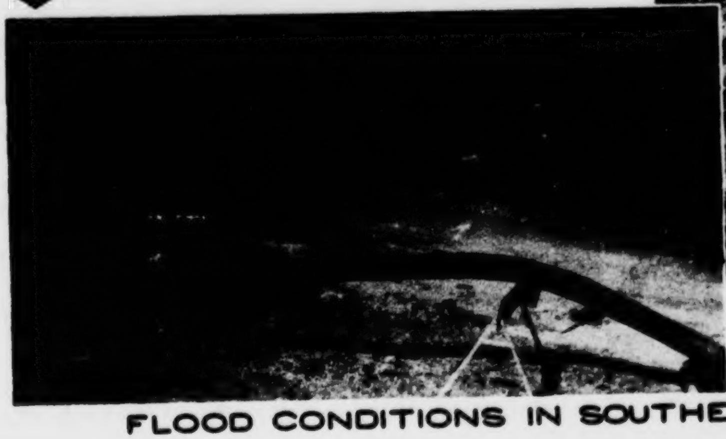
FIGURE 62



STATION D-5
SOUTH SIDE TRUNK
SIPHON INLET

FLOOD PREVENTS ACCESS AND MAINTENANCE WITH RESULTING SILTATION AND RESTRICTION OF FLOW THROUGH SIPHONS

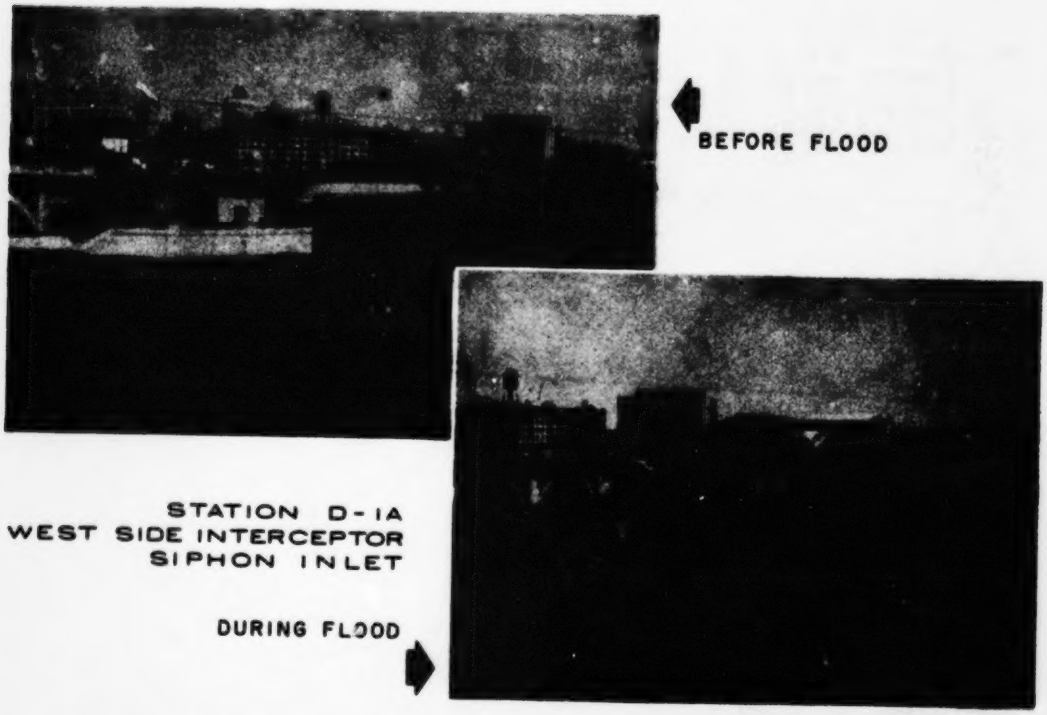
SURCHARGE SANITARY SEWERS REQUIRE RELIEF PUMPING INTO STORM SEWERS



"BLOWN" MANHOLE CAUSED BY SURCHARGING

FLOOD CONDITIONS IN SOUTHEAST DES MOINES

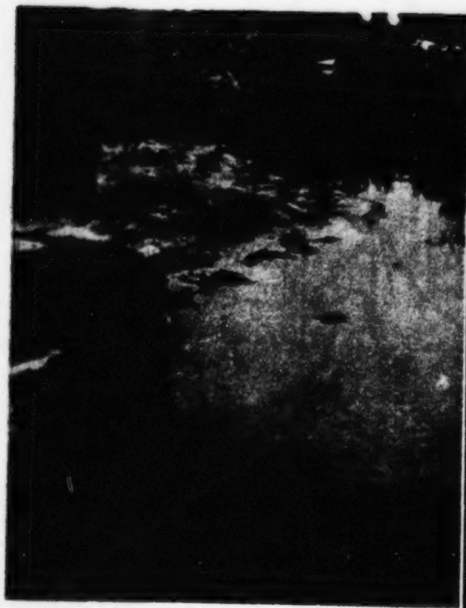
FIGURE 63



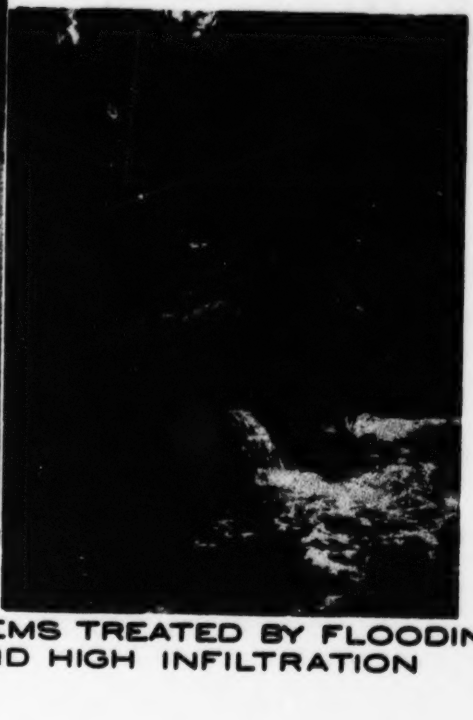
BEFORE FLOOD

STATION D-1A
WEST SIDE INTERCEPTOR
SIPHON INLET

DURING FLOOD



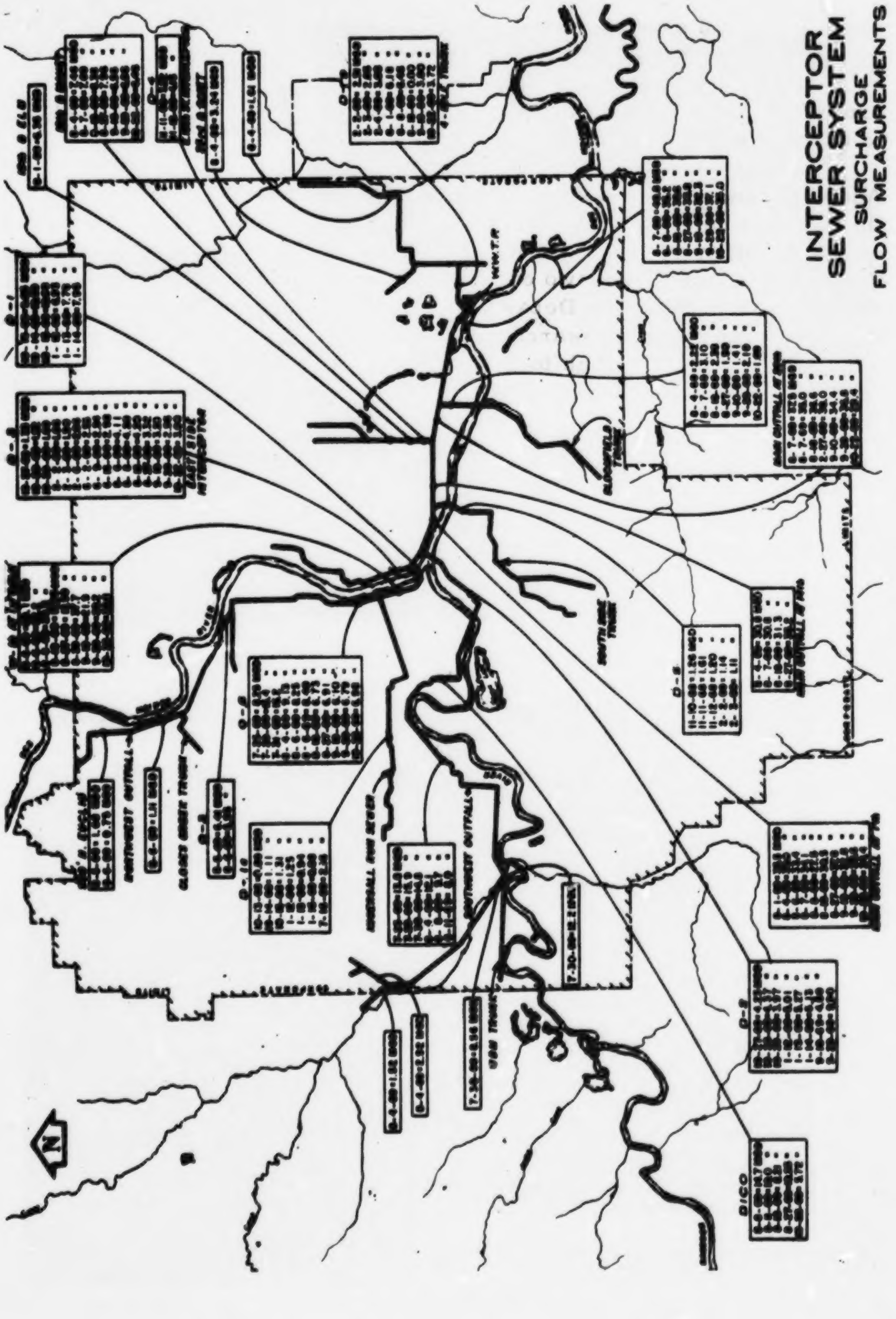
FLOOD WATERS VORTEX
INTO SIPHON CHAMBER



SURCHARGE CAUSES BREAK IN
30"; CLOSES CREEK SEWER

PROBLEMS TREATED BY FLOODING
AND HIGH INFILTRATION

FIGURE 64



INTERCEPTOR SEWER SYSTEM SURCHARGE FLOW MEASUREMENTS

FIGURE 65

monitoring and sampling data. It did, however, reduce the field engineering time available for management and data analysis. It should be recognized that the lack of up-to-date records is not an uncommon problem. Care should be taken in developing future study programs to ascertain the completeness of system records.

Another item regarding available records, one which did not affect this project but which nonetheless deserves comment, is the sources and availability of river quality data. As noted in Section VIII, river data was obtained from two outside sources, the State Hygienic Laboratory through the State Department of Health and the Iowa State University Engineering Research Institute. Each of the above have ongoing sampling programs which to a considerable degree overlap each other.

It is recognized that different levels of data collection is required of different types of projects. A central source for obtaining this data, such as the Federal Storet system, should be provided to assist in disseminating this type of information.

SECTION XII

SUMMARY OF COST ESTIMATES

Estimated project costs for the systems described for Plans A, B-1 and B-2 are shown in Table 23. Project costs are construction costs, shown in previous sections, plus an allowance for engineering services, legal, financial and administrative costs. For Plan A, the cost of separation is broken down into systems corresponding to those described in the treatment section.

Annual costs for the the three plans are shown in Table 24. The average annual principal and interest is based on a 20 year debt retirement at 6 percent interest. Operation and maintenance costs are tabulated and the total average annual costs are given for each system. The cost of maintaining the collection systems are not included in this comparison since this should be essentially the same whether separate or combined systems are used. Nor are the estimated costs for the wastewater treatment plant improvements included. Operating costs are the average for the 20 year financing period.

All of the plans described would eliminate or substantially reduce the overflow pollution load to the river. Average annual costs for Plan B-1, in which all overflows are contained and treated is approximately 9 percent less than complete separation. Plan B-2, which allows infrequent overflows of highly diluted combined wastes and greatly reduces the overflow pollutional load, would cost about half of complete separation.

The Closes Creek System illustrates how treatment of overflows and separation can be used effectively together in some cases, to minimize the cost of eliminating combined sewer overflows. For that system as a whole, the average annual cost of $191,800 for total separation is comparable to $289,100 annually for interception and treatment and $149,700 annually where both treatment and separation are proposed.

In the East 18th Street System, diversion and treatment of diluted combined sewage would eliminate a major problem, i.e., adding voluminous quantity to a heavy concentration of industrial waste, and does so at considerably less than the cost of separating the combined sections of that watershed. In other systems, such as the Freeway System on the west side, separation was the only practical solution.

It is apparent that each case must be evaluated on its own merits to determine whether separation or treatment of combined sewer

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overflows is the most feasible means of control. The nature and use of the receiving waters, public health, aesthetics and public acceptance, as well as financial considerations, will influence the selection of overflow control measures.

Plan C, which is essentially Plan B-1 with additional treatment facilities for all storm water discharges, is not included in the tables. The estimated project cost for storm water treatment alone is $154,905,000 and the estimated operating cost is $415,700 annually. Adding the costs of Plan B-1, the annual costs of Plan C would be as follows:

P&I	$15,103,700
O&M	525,300
Total	$15,629,000/year

TABLE 23

SUMMARY OF PROJECT COSTS

		Plan A	Plan B-1	Plan B-2
1.	Closes Creek System			
	Retardation Basins	$ -	$ 1,840,000	$ -
	Separation	$ 2,200,000	36,000	$ 623,000
	Prospect Road Impoundment	-	1,095,000	899,000
2.	West Side Interceptor			
	Separation	3,200,000	156,000	156,000
	Storm Box Extension	-	1,693,000	432,000
3.	Ingersoll Run System	4,400,000	4,400,000	29,000
4.	East Side System	6,500,000	1,907,000	1,279,000
5.	East 18th Street System	1,900,000	427,000	427,000
6.	South Side Trunk	2,200,000	121,000	121,000
7.	Scott St. Lift Station & Storm Outfall	-	5,134,000	4,298,000
8.	Case Lake Treatment Complex	-	1,534,000	1,711,000
9.	Miscellaneous Separation	900,000	-	-
	Total	$21,300,000	$18,343,000	$9,975,000

TABLE 24

SUMMARY OF ANNUAL COSTS

	Plan A	Plan B-1			Plan B-2		
	Total	P&I*	O&M**	Total	P&I*	O&M**	Total
1. Closes Creek System							
Retardation Basins	-	160,400	7,600	168,000	-	-	-
Separation	191,800	3,100	-	3,100	54,300	-	54,300
Prospect Road Impoundment	-	95,500	22,500	118,000	78,400	17,000	95,400
2. West Side Interceptor							
Separation	279,000	13,600	-	13,600	13,600	-	13,600
Storm Box Extension	-	147,600	-	147,600	37,700	-	37,700
3. Ingersoll Run System	383,6000	383,600	-	383,600	2,500	-	37,700
4. East Side System	566,700	166,300	-	166,300	111,500	10,000	121,500
5. East 18th St. System	165,700	37,200	4,400	41,600	37,200	4,400	41,600
6. South Side Trunk	191,800	10,500	3,100	13,600	10,500	3,100	13,600
7. Scott St. Lift Station & Storm Outfall	-	447,600	48,000	495,600	374,700	44,000	418,700
8. Case Lake Treatment Complex	-	133,700	24,000	157,700	149,200	28,000	177,200
9. Miscellaneous Separation	78,500	-	-	-	-	-	-
	$1.857,100	$1,599,100	$109,600	$1,708,700	$869,600	$106,500	$976,100

* Average Principal & Interest, 20 years @6%

** Annual Operation & Maintenance

SECTION XIII

ACKNOWLEDGEMENTS

Only through the cooperation and assistance received from the City of Des Moines was the completion of this project possible. Mr. Leo Johnson, Director of Public Services, and the entire staff of the City Public Works Department gave generously and enthusiastically of their time and knowledge. Contract analytical services were provided by the Engineering Research Institute, Iowa State University under the direction of Dr. E. R. Baumann and by the State Hygienic Laboratory, University of Iowa under the direction of Senior Chemist, Lauren G. Johnson.

Numerous other organizations and agencies provided valuable assistance in the project, including the Central Iowa Regional Planning Commission, the U. S. Weather Bureau, and the cities of Urbandale and West Des Moines, to mention but a few.

The support of the project by the Environmental Protection Agency and the guidance and help provided by Mr. Ralph G. Christensen, Project Officer, and Mr. W. A. Rosenkranz, Director of the Municipal Pollution Control Division and his staff; and Mr. Richard Field, Chief, Storm and Combined Sewer Technology Branch is acknowledged with appreciation.

SECTION XIV

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SECTION XV

GLOSSARY

Pollution - the act of degrading or reducing the usefulness of a system, such as surface waters, by the introduction of physical, chemical or biological changes and/or additions to that system.

Sanitary Sewage (or Wastewater) - is the water carried wastes which originate in the sanitary conveniences of a dwelling, business establishment, factory, or institution.

Storm Water - is the excess water running off from the surface of a drainage area during and immediately after a period of rainfall.

Infiltration - is the groundwater which gains entrance into the sewers through joints, improper connections, etc., as differentiated from surface runoff.

A Sewer - is a pipe or conduit used for the purpose of conveying sewage. There are three general classifications of sewers:

a. A Sanitary Sewer is one designed to carry sanitary sewage only. In many cases, it will also carry industrial wastes produced in the area it serves.

b. A Storm Sewer carries storm runoff and similar waters not including sanitary sewage.

c. A Combined Sewer is designed to carry domestic sewage, industrial wastes, and storm runoff in a single conduit.

The term Sewerage (or Wastewater Facilities) - is used to designate a system of sewers and appurtenances for the collection, transportation, and pumping of sewage and industrial wastes.

Wastewater Treatment Plant - is a comprehensive term encompassing the arrangement of devices and structures for treating sewage and industrial wastes and sludge.

A Main Sewer - is one to which one or more branch sewers are tributary.

An Intercepting Sewer - is a sewer which receives dry weather flow from a number of transverse sewer outlets and frequently additional predetermined quantities of storm water (if a combined system) and conducts such waters to a point for treatment or disposal.

An Outfall Sewer - is a sewer which receives the sewage from a collecting system and carries it to a point of final discharge.

Sewage Treatment (or Wastewater Treatment) - refers to any artificial process to which sewage is subjected in order to remove or alter its objectionable constituents so as to render the sewage less dangerous or offensive.

Sewage Disposal (or Wastewater Disposal) - applies to the act of disposing of sewage by any method. It may be done with or without previous treatment of the sewage.

Biochemical Oxygen Demand (BOD) - is the quantity of oxygen utilized in the biochemical oxidation of organic matter in a specified time and at a specified temperature. It is not related to the oxygen requirements in chemical combustion, being determined entirely by the availability of the material as a biological food and by the amount of oxygen utilized by the microorganisms during oxidation.

Solids - the solid content of a sewage consists of those in the settleable, suspended, dissolved and total form. The total solids represents the sum of the suspended and dissolved contents. The total solids and suspended solids are further divided into volatile and nonvolatile for the purpose of differentiating between the organic and inorganic content. Settleable solids are those readily amenable to settling irrespective of their size.

COD - chemical oxygen demand, the measurement of the total quantity of oxygen required to chemically oxidize all organic compounds, with a few exceptions, to carbon dioxide and water regardless of the biological assimilability of the substances.

Precipitation - includes all forms, such as rain, snow, sleet, etc.

Design Rainstorm - a selected rainstorm of an area, involving the duration, intensity and recurrence interval of the storm, for use as a design basis.

Runoff - the flow of waters from precipitation or thaw incidents from gutters into street inlets or from other connections into storm or combined sewer systems.

Combined Sewer Overflow - the discharge into receiving waters of liquid wastes from combined sewers through outlet structures which regulate the amounts of flows either carried by trunk sewers or disposed of into such receiving water resources.

ABBREVIATIONS

BOD	Biochemical Oxygen Demand, 5-day
DO	Dissolved Oxygen
COD	Chemical Oxygen Demand
TSS	Total Suspended Solids
VSS	Volatile Suspended Solids
GCPD	(gal/capita/day) - gallons per capita per day
mg/l	Milligrams per liter
ug/l	Micrograms per liter
ppm	Parts per million
psi	Pounds per square inch
CF/MG	Cubic feet per million gallons
MGD	Million gallons per day
GPD	Gallons per day
WWTP	Wastewater Treatment Plant
JCU	Jackson Candle Units

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APPENDIX A

GENERAL INFORMATION AND DESCRIPTION OF SAMPLING STATIONS, MONITORING STATIONS, OVERFLOW POINTS

To avoid repetition in the main body of the report, general information and detailed descriptions of the various samplings and monitoring stations and overflow points are all presented in this Section. For ease of reference, the individual stations and points are grouped by drainage area. Contributing areas, populations, and other pertinent monitoring station information is summarized in Table 4. Sampling and monitoring point locations are shown in Figures 13 and 14.

WEST SIDE INTERCEPTOR SEWER SYSTEM (Areas I and IA)

Dry Weather Stations: D-1, D-1A, D-1B.

Wet Weather Stations: W-1, W-1A.

Monitored Overflow Stations: O-2, O-3, O-4, O-5, O-6, O-7, O-8, O-8A.

This system, serving a total of 9,608 acres and 89,100 population, is the largest system from the standpoint of both wastewater flows and combined sewer overflows. The area is shown in Figure 13 and includes all the downtown area north and west of the Raccoon and Des Moines Rivers. About 24 percent of the total area is served by combined sewers. These are located mostly in the downtown area, along the river front and in the Ingersoll Run (D-1A) area. A few combined sewers were found in the upper Closes Creek Area, as shown in Figure 14. Practically the entire system, however, suffers from excessive infiltration - most of which is believed to be from building footing drains. Following is a description of the individual sampling and monitoring points.

1. Station D-1

Station D-1 was the terminal point of the West Side Interceptor Sewer and is located at the Scott Street siphon outlet chamber. The siphons are 30-inch and 42-inch pipe and for part of the route across the river are encased in the Scott Street Dam. During the October, 1968 dry weather sampling, samples were taken manually and flows were measured with a current meter. The measuring section was below the confluence of the two siphon outlet channels. For the January resampling, weirs were constructed in each of the two siphon channels and float recorders were used to record head. Samples were collected with an automatic sampler. Figure 6 shows the setup and subsequent problems which occurred at this station.

2. Station D-1A

Station D-1A is located on the Ingersoll Run Sewer System at 22nd and High Streets. This sewer is tributary to the West Side Interceptor via the Walnut Street Sewer. This particular site was chosen for monitoring because a major overflow (O-8A) is located immediately below it. At the sample site, the sewer is a 5' by 13' box section. Below the overflow, the line is 42-inch diameter, two-ring brick sewer. The service area is primarily residential housing with some commercial development along Ingersoll and Grand Avenues. The area is also quite hilly. Dry weather flows for both sample periods were measured with a rectangular weir (see Figure 6) and a float recorder. The station was manually sampled during the first period. During the second period, an automatic sampler was lowered through a manhole and was set on a platform similar to the one used for the float recorder.

3. Station D-1B

Station D-1B was located on the 30" Closes Creek Trunk Sewer at an abandoned pump station near the intersection of Harding and Prospect Roads. This station was sampled only during the first dry weather schedule. It was not considered significant enough to the overall program to resample this station for quality data alone. Flow data from the first sampling was used. Because the station was not resampled, a data sheet was not included in this text.

4. Stations W-1 and W-1A

Station W-1 is the same location as D-1. The purpose of this station was to record water level in attempt to correlate head loss across the siphon with flow in the siphon.. Station W-1A was the upstream station and was located in the West Side Interceptor siphon inlet chamber. Bubbler units were used for water level recording at both stations. Both stations are subject to frequent extended surcharging, W-1 from backup in the main outfall and W-1A from the excessive infiltration previously discussed.

Also, a major overflow is located at W-1A and it was active during much of the spring and summer. An attempt was made to correlate water levels at W-1A with measured flows through the overflow, which sometimes acts as an orifice. The results were erratic, and at best provided only an approximation of the head-discharge relationship for this overflow.

5. Station W-1B

Station W-1B was an alternate flow measuring point to Station W-1A and was located at the first manhole downstream from West 1st and Elm Streets. Flows were computed from depth and current meter observations.

6. Station O-2

Station O-2 was located at the apex of the 1700 acre Closes Creek drainage area. The drainage area is mainly residential, hilly, and heavily wooded in some sections. Most of the flow measured at this station was storm sewer discharge and overland runoff. There are however, about 107 acres around the upper perimeter of the basin which are combined. Flows at Station O-2 were determined by stage discharge relationship. Fortunately, the sampling point was at the site of a permanent control which minimized discharge rating curve shifts. Sufficient low flow measurements were made to establish the lower section of the rating curve and then high water measurements were made as often as possible during runoff periods to provide a basis for projecting the curve. All sampling was done with the automatic samplers. To facilitate sampling, a special platform was suspended from a foot bridge over the creek. Figure 10 shows the O-2 control.

7. Station O-3

Station O-3 was an overflow point at the upper end of the West Side Interceptor Sewer. The station was located on the 30-inch sewer near Prospect and Hickman Roads at a point below the junction of the Closes Creek Trunk and the Northwest Outfall. At this point an overflow weir has been effected by the removal of the upper part of the sewer barrel. The overflow section is enclosed in a concrete box. Overflow is to a small drainage way leading directly to the river a short distance to the east. Initially, this overflow was monitored by periodic inspection of a stick gage. This information indicated that overflow did occur during periods of significant runoff. Later in the project, a bubbler unit was installed and samples collected with an automatic sampler.

8. Station O-4

Station O-4 was an overflow on the West Side Interceptor at the intersection of 2nd Avenue and Franklin. The overflow is equipped with a flood gate to prevent backup of river water into the sewer system. When operating, the overflow releases surcharge from the West Side

Interceptor and from the Franklin Street Sewer which is tributary to the interceptor. The actual overflows are weirs in the respective sewers. Both weirs are relatively high. A stick gage was used to determine overflow frequency and from this information it was determined that additional monitoring was not warranted.

9. Station O-5

Station O-5 is a major overflow on the West Side Interceptor. It is located within the IPALCO power plant complex at West 1st Street and Grand Avenue. The main sewer and overflow are shown in Figure 15. From inspection of the overflow and periodic inspection of a stick gage installed therein, it was determined that this point warranted continuous monitoring and consequently a bubbler setup was constructed. The bubbler recorded incidence of overflow and provided an upstream head reading for hydraulic gradient computations to Station O-6, 451 feet downstream. During high river stage, the overflow was submerged almost constantly. During normal dry weather operation, overflow occurs only during peak periods. Because of the relative low elevation of the overflow and its close proximity to the river, this point is a prime suspect as the point of access for fish found in the raw flow at the wastewater treatment plant. It is not however, a source of river water getting into the sewer system as had been speculated early in the extended high water period. By monitoring the level in the sewer and cross-checking with river elevations, it was determined that flow was always in the direction of the river.

10. Station O-6

Station O-6 is also an overflow on the West Side Interceptor in the area of West 1st and Grand. As noted above, it is 451 feet downstream from Station O-5 and was a point used for determination of flow by the hydraulic gradient method. (A table was developed for this section of sewer to convert head differential and flow depth to discharge.) The overflow is a brick wall constructed in a broken out section of the pipe (see Figure 15). The wall extends to within a foot of the top of the pipe and monitoring indicated that overflow never occurred at this point. This was not unexpected considering the major overflow immediately upstream. All sampling for the O-5/O-6 area was done at Station O-6. Sampling was done with an automatic sampler which was lowered into the manhole.

11. Station O-7. (Also O-7A and O-7B)

The O-7 stations were located on the West Side Storm Box, a 5 foot by 13 foot concrete conduit located outside the floodwall and along the west bank of the Des Moines River from Scott Street to Grand Avenue. This box is the recipient of overflows from several sewers normally tributary to the West Side Interceptor. The tributary sewers are in Grand Avenue, Locust Street, Walnut Street, and Elm Street. In addition, the box receives all storm and combined flow from the Birds Run Sewer. To accommodate the high peak flows which occur, mainly as a result of the large Birds Run Sewer, the upper end of the box is equipped with pressure release flap gates which permit discharge from both ends of the box. The lower end of the box siphons under the confluence of the Des Moines and Raccoon Rivers and discharges through the face of the Scott Street Dam. This box may also be used for lowering the pool of the impoundment behind Scott Street Dam. Wooden sluice gates are provided at the lower end of the box for this purpose.

Three bubbler stations were located along the length of the box in an attempt to measure flow by hydraulic gradient. The locations were: (1) O-7 at the lower end of the box; (2) O-7A about 990 feet above the end of the box; and O-7B at the upper end of the box. This procedure did not provide meaningful correlations due to excessive leakage from the river along most of the length of the box and considerable debris inside the box. Figure 11 illustrates these problems. Station O-7A was sampled on several occasions to obtain comparative data for highly diluted overflow waters.

12. Station O-8. (Also O-8A)

Station O-8 is the outlet of the Ingersoll Run combined sewer overflow at 17th and Railroad Avenue. Flows reaching this point come from the Ingersoll Run overflow at 22nd and High Streets (Stations D-1A and O-8A) and from an unknown number of storm sewers intercepted in between. The actual overflow at O-8A is a 21.5-foot broad crest weir in the converging section between the 5-foot x 13-foot Ingersoll Run Box and the 42-inch combined sewer downstream. Results of early samplings, and the probability that flooding and surcharging at other major overflow points would require this station to be a major source of information, prompted the establishment of Station O-8A as a cross-check on the data obtained from the more dilute flows at Station O-8. Sampling at both stations was done with an automatic sampler. Water levels were obtained via bubbler installations. For Station O-8, flows were obtained by stage-discharge relationship. A rock and debris riffle immediately below the station served as a satisfactory control for the flow range measured (see Figure 10).

SOUTHWEST OUTFALL SEWER (Area II)

Dry Weather Station	D-2
Wet Weather Station	W-2

1. Station D-2

This system serves the western section of Des Moines and the cities of West Des Moines, Windsor Heights, Clive and Urbandale. The contributing area encompasses 15,720 acres with a population of about 54,700. The collection systems served are designed as separate sanitary sewers; however, flow studies made during this project indicate the systems carry excessive infiltration and/or foundation and roof drain waters. The contributing area is primarily residential and light commercial development. Only about 1 percent is industrial. The terrain is rolling and has extensive foliage.

There are no known overflows in the D-2 system with exception of a manually operated bypass gate on the outfall at 17th Street. This gate is to be used only if maintenance is required on the sewer or the siphons.

The D-2 sample station was in the Raccoon River siphon outlet structure near Southwest 7th and Indianola Avenue. Flow was measured over a 5-foot rectangular weir at the downstream end of the structure. Samples were collected manually during the October 1968 dry weather composites. Resampling in January was done with automatic samplers. Figure 9 shows several photos of the D-2 setup.

2. Station W-2

Station W-2 was the same as Station D-2. For continuous flow monitoring, the weir installed for dry weather sampling was left in place and a bubbler unit installed for water level monitoring. While monitoring, it was determined that the station is subject to frequent extensive surcharging, apparently from the main outfall sewer. Because of this situation, the original plan to install a bubbler at the siphon inlet and determine flow from head loss across the siphon was abandoned. In lieu thereof, sewer flow measurements were made by current meter at several upstream points during the "wet" dry weather period. These flows are shown in Figure 65 and led to the previous mentioned conclusion that the system is subjected to excessive infiltration. Whenever possible, daily "wet" dry weather flows were computed for comparison with the point flow and dry weather flow measurements. The data obtained confirmed the point flow measurements and the conclusion that excessive extraneous flows have access to this system.

EAST SIDE INTERCEPTOR SEWER (Area III)

Dry Weather Station	D-3
Wet Weather Station	W-3
Monitored Overflow Points	O-9, O-10

1. Station D-3

This system serves the east side downtown area and a north-south corridor along the east side of the Des Moines River. The service area is about 2,240 acres with an estimated contributing population of 16,400. About 24 percent of the collection system is combined sewers and there are two overflows to the Des Moines River, one at O-9, the Birdland Pump Station, and the other at O-10, the discharge from the East Side Storm Box. Actually, the storm box receives overflow from the East Grand and East Locust Street sewers in the downtown area. The Area III terrain is relatively flat except for a corridor along the river. Except for the east side downtown area, the sanitary watershed is mainly residential development. Industrial development occupies only 2.7 percent of the total area.

The D-3 sample station was a manhole at East 1st and Raccoon Streets. A 30-inch Cipoletti weir was constructed in the 48-inch brick sewer and a float recorder used to determine head. Samples were collected manually during the October, 1968, sampling period and with an automatic sampler during the February, 1969, resampling.

2. Station W-3

Station W-3 is at the same location as D-3. The weir used for dry weather measuring was left in place for use during non-surcharge conditions. A bubbler unit was used for recording head readings. The unit was kept in service through the "wet" dry weather period for recording of surcharge water surface elevations. Because of its relative elevation, this station was affected less by surcharge than several of the other wet weather stations. Valuable "wet" dry weather daily flow data was obtained at this station.

3. Station O-9

This overflow is at the Birdland Pump Station. Stick gate monitoring indicated that overflow does occur, although apparently only during significant rainfall. The overflow is gated so can be maintained closed as long as upstream surcharging does not create a public health hazard or cause property damage. This station was not monitored except for periodic inspection of the stick gage.

4. Station O-10

The O-10 station was the East Side Storm Box outlet. This conduit is identical to Station 0-7, the West Side Storm Box, and is used for the same purposes, including bypassing of river flow around the Scott Street Dam. During the course of the project, this box was never able to be monitored. Flood water prevented monitoring for much of the rainy period. After recession of the high water, it was determined that the box was 2/3 to 3/4 filled with sediment, particularly at the upper end in the vicinity of the Grand and Locust Street overflows to the box. Also, this box was in use as a river bypass around Scott Street Dam for the duration of the study. It is believed that this box is a major overflow and any remedial program should include collection of flows therein.

EAST 18TH STREET INTERCEPTOR (Area IVA and IVB)

1. Dry Weather Station D-4

This system serves the east-central area of Des Moines covering 3,024 acres with an estimated contributing population of 17,300. Much of the industry in the City is in this area. The sewers in the collection system are mostly separate sanitary sewers, although a few combined sewers (5 percent of the area) are known to exist in the northwestern part of the area. There are no known points of over-flow in the system. The interceptor discharges into the main outfall sewer near 18th and Maury Streets.

The D-4 sample station was in a manhole just above the connection to the main outfall. A 36-inch rectangular weir was constructed in the 66-inch brick interceptor sewer and a float recorder used to determing head. Samples were collected with an automatic sampler. This station was only sampled during the initial dry weather sampling period. This station was not resampled, even though the initial data is questionable, because it was learned that the industries contributory to the system would soon be improving their inplant waste treatment systems in accordance with a new industrial waste ordinance and control program. Also, the results of individual industrial waste sampling by the City's staff were made available for our evaluation.

SOUTH SIDE TRUNK (Area VII)

Dry Weather Station	D-5
Wet Weather Stations	W-5 and W-5A
Overflow Station	O-13

This system serves 2,061 acres with an estimated population of 15,300 and is located in the south central part of Des Moines. The area served is an older area of the City and is primarily residential. About 7 percent of the collection system is combined sewers.

The system terminates at Southeast 9th and Jackson Streets in a two-barrel siphon (14 and 20 inch) under the Des Moines River. Siphon discharge goes into the main outfall at Southeast 9th and Railroad. The only overflow in the system is at the siphon inlet structure.

1. Station D-5

The D-5 sample station is in the siphon inlet structure. One of the siphon channels was blocked off and a weir constructed in the other channel. A float recorder was used to determine head over the weir. Sampling was done manually during the November, 1968, dry weather sampling and with automatic samplers in the February resampling.

This station has some very severe maintenance problems. Figure 63 shows the station during flood conditions and also one of the siphon inlet channels filled with sediment. Similar conditions were noted throughout the study period.

2. Stations W-5 and W-5A

Station W-5 was identical to D-5 except that a bubbler unit was used instead of a float recorder. Station W-5A was the siphon outlet structure on the north side of the river. A bubbler unit was installed here in the hope of being able to correlate head loss across the siphon with incoming wet weather flow. Backup from the main outfall, to which this line is tributary, affected the head readings during wet weather and throughout all "wet" dry weather period. This surcharge caused almost constant overflow just upstream of the siphon during the wet and "wet" dry weather periods. Flow computations provided erratic results. No meaningful correlations could be made relative to the amount of rainfall required to produce overflow at this point.

3. Station O-13

Station O-13 is the overflow point referred to above. This is a major overflow. As noted above, overflow during spring and summer was almost continuous, even during relatively dry periods. Because of the conditions already noted, the volume of overflow could not be determined. Some sampling was done to provide strength data for comparison with the dry weather sampling.

MAIN OUTFALL AND WWTP (Area IX)

Dry Weather Station	D-6
Overflow Station	O-14

The Des Moines wastewater treatment plant serves all of the Des Moines and the surrounding metropolitan area except: (1) Highland Hills, a small development south of Army Post Road at the south edge of the City; (2) Pleasant Hill, a small community east of the City; and (3) the Urbandale Sanitary District which is a small segment of the City of Urbandale. The area served by the Des Moines Plant is about 46, 167 acres and has an estimated contributing population of 239, 700.

1. Station D-6

The D-6 sample station was at the bar screen channel at the treatment plant. Samples were collected and analyzed by the treatment plant chemist. Flow was determined from the raw sewage flow totalizer. Unfortunately, the plant was undergoing expansion construction during the dry weather sampling period and was unable to handle the total raw flow. Therefore, the data obtained for that period cannot be used for comparison with the in-system dry weather data. To compensate for this problem, plant operation records for the 3-months period of August-September-October, 1969, were obtained and evaluated.

2. Station O-14

Station O-14 was the WWTP raw sewage bypass. Originally it was intended to monitor this point continuously. Two physical problems prevented accomplishment of that goal and limited actual monitoring to a short period in March and approximately two months record in late summer and early fall. Initial delay in monitoring this point was caused by construction of a flood protection levee and flood gate at this location. Within a few days after the site became available for monitoring, the first of several periods of high river stage occurred. Figure 62 illustrates the flood problem. A weir and bubbler was finally installed after recession of high water. Considering the data obtained after the bubbler was installed, and recognizing that bypassing at this point was unlikely during the period of missed record, it appears that this point was not a major overflow during the study period. It is likely that upstream overflow, caused by the surcharged main outfall sewer, minimized occurrence of bypassing at this location.

CORNELL AND AURORA STORM SEWER OUTLET

1. Dry Weather Station D-7

This storm sewer was reported to have sanitary connections and therefore was included in the initial dry weather sampling. The sample point was at the outlet of the storm sewer in an open ditch at Cornell and Aurora Avenues. A wooden bulkhead and weir were constructed in the ditch and a float recorder used to determine head over the weir. Samples were collected with an automatic sampler. The station was sampled during the initial dry weather work only, since observation during setup and sampling and the laboratory data indicated an absence of sanitary flow.

FRALEY DITCH STORM SEWER OUTLET

1. Dry Weather Station D-8

This was another storm sewer which was included in the dry weather sampling program because it was thought to carry sanitary wastes. The station location was at East 30th and Court Street. A 90° V-notch weir was installed at the outlet of the storm sewer. A float recorder was used to determine head and samples were collected with an automatic sampler.

During the November-December dry weather sampling, it was determined that the station did receive a significant quantity of industrial waste. Prior to the re-sample period, it was learned that the City would be including this sewer in its industrial waste control program; therefore, the station was not re-sampled. Instead, data from the City's program was obtained. From the City's data it appeared that the normal flow is free of sanitary wastes but did contain a significant quantity of industrial waste. These wastes were subsequently removed from the system.

THOMPSON AVENUE STORM SEWER

1. Storm Runoff Station S-1

The station is located at the outlet of the Thompson Avenue Storm Sewer at the Birdland Park Marina (see Figure 14). The drainage area served is 310 acres. Most of the area is older residential area with considerable open park space along the natural waterway. Discharge from this point flows in to the Birdland Park Marina and thence into the Des Moines River.

The station was monitored by constructing a weir and bubbler setup at the outlet of the box. Except for periods of high river stage (see Figure 62), satisfactory continuous record was obtained. This station was a key station in the rainfall-runoff study.

CUMMINS PARKWAY STORM DRAINAGE

1. Storm Runoff Station S-3

The Cummins Parkway watershed is located in a rolling hilly area near the western Des Moines city limits (see Figure 14). The 356 acre area is almost entirely residential and has considerable open grassy area. Discharge from this station flows into Walnut Creek near 63rd and Grand.

The station was monitored by constructing a weir and bubbler setup at the outlet of a box culvert. The setup is shown in Figure 10. The station was a key station in the rainfall-runoff study.

20TH STREET STORM SEWER

1. Storm Runoff Station O-11

The O-11 station was originally expected to be a combined sewer overflow point, hence the "O" designation. During the study, however, it was determined that the flow at this point was separate storm flow. The contributing area to Station O-11 is about 1170 acres located in the north-central part of the City and includes considerable industrial and commercial area. The outlet of the 4' by 5' storm sewer is at the upper end of Dean Lake at E. 22nd and Dean Avenue (see Figure 14). Monitoring was accomplished by installation of a weir and bubbler unit. Sampling was done with automatic samplers. Figure 7 shows the monitoring and sampling setup. The 55-gallon drum is the sampler housing used to prevent vandalism.

APPENDIX B

TABULATION OF MONITORING AND SAMPLING RESULTS

Wet and Dry Weather Sanitary Flows

Combined Sewer Overflows

Storm Water Discharges

Note:

Caution should be exercised in interpreting the information herein. Data given for samples collected and analyzed is not necessarily for a single complete runoff from rainfall or snow melt. Grab samples are considered point values, although an arbitrary duration may have been assigned for extension to pounds. Hourly dry weather sanitary flows are shown graphically in the Figures at the back of this Appendix.

EXPLANATION OF TABLE

The following explanations are given for column headings:

TIME: Given in Military Time System, beginning time for composite samples.

TYPE: Type designation establishes type of sample and source of flow. Code established for type of sample is:

C - composite sample

S - single grab sample

Code established for source of flow is:

D - dry weather sanitary flow

W - wet weather sanitary or combined flow

R - overflow or runoff due to rainfall

S - overflow or runoff due to snow melt

DURATION:	Time, in hours, for which composite sample was made up.
FLOW:	Average rate of flow, in cubic feet per second, for the duration of composite sample, or rate of flow at the time of grab sample.
MG/L:	Strength of sample in milligrams per liter.
LBS:	Pounds of pollutants for the duration indicated.
N.A.:	No analysis made.

DES MOINES, IOWA, STORM WATER POLLUTION CONTROL STUDY
STATION D-1 (W-1) E. SIDE INTERCEPTOR AT SCOTT

FWPCA CONTRACT NO 14-12-402

ITEM	DATE	TIME	TYPE	DUR* HOUR	FLOW MGD	BOD MG/L LBS	SUSPENDED SOLIDS TOTAL MG/L LBS	SUSPENDED SOLIDS VOLATILE MG/L LBS	SUSPENDED SOLIDS VOLATILE PER CENT	NITROGEN AMMONIA MG/L LBS	NITROGEN NITRITE MG/L LBS	NITROGEN NITRATE MG/L LBS	PHOSPHATES TOTAL MG/L LBS	PHOSPHATES SOLUABLE MG/L LBS	CHLORIDES MG/L LBS	CHROMIUM UG/L LBS
1	10/13/68	8 0	CD	24.00	9.0	-N.A.-	-N.A.-	-N.A.-		-N.A.-	-N.A.-	-N.A.-	-N.A.-	-N.A.-	-N.A.-	-N.A.-
2	10/14/68	8 0	CD	24.00	10.5	-N.A.-	-N.A.-	-N.A.-		-N.A.-	-N.A.-	-N.A.-	-N.A.-	-N.A.-	-N.A.-	-N.A.-
3	10/15/68	8 0	CD	24.00	11.0	-N.A.-	-N.A.-	-N.A.-		-N.A.-	-N.A.-	-N.A.-	-N.A.-	-N.A.-	-N.A.-	-N.A.-
4	12/12/68	23 0	SD	1.00	N.A.	275.00	-N.A.-	-N.A.-		27.60	.20	1.44	37.50	29.20	-N.A.-	-N.A.-
5	12/13/68	9 0	SD	1.00	N.A.	218.00	-N.A.-	-N.A.-		32.30	.36	3.31	24.00	21.80	-N.A.-	-N.A.-
6	1/12/69	9 0	CD	24.00	7.0	194.00 11312.1	216.00 12595.0	199.00 11603.7	92	24.21 1411.7	.09 5.2	2.60 151.6	19.00 1107.9	12.00 699.7	-N.A.-	-N.A.-
7	1/13/69	9 0	CD	24.00	7.8	204.00 13254.7	277.00 17997.8	260.00 16893.2	94	54.94 3569.7	.07 4.5	0.00 0.0	19.48 1265.7	8.65 562.0	-N.A.-	-N.A.-
8	1/14/69	9 0	CD	24.00	8.0	241.00 16060.2	269.00 17926.2	234.00 15593.8	87	25.03 1668.0	.09 6.0	0.00 0.0	15.90 1059.6	7.77 517.8	-N.A.-	-N.A.-
9	5/21/69	9 0	SD	1.00	N.A.	130.00	-N.A.-	-N.A.-		8.66	.34	2.49	-N.A.-	-N.A.-	-N.A.-	-N.A.-
10	5/21/69	10 0	CW	2.00	N.A.	69.00	449.00	114.00	25	-N.A.-	-N.A.-	-N.A.-	-N.A.-	-N.A.-	-N.A.-	-N.A.-
11	5/21/69	12 0	CW	4.00	N.A.	28.00	595.00	132.00	22	1.31	.04	2.88	5.10	3.24	-N.A.-	-N.A.-
12	5/21/69	16 0	CW	7.00	N.A.	90.00	256.00	123.00	48	4.01	.43	1.86	10.14	5.40	-N.A.-	-N.A.-
13	5/22/69	6 0	CW	2.00	N.A.	62.00	113.00	64.00	57	-N.A.-	-N.A.-	-N.A.-	-N.A.-	-N.A.-	-N.A.-	-N.A.-
14	6/26/69	11 0	SW	1.00	N.A.	110.00	281.00	149.00	53	15.40	.02	.11	15.80	10.60	-N.A.-	-N.A.-
15	6/26/69	1740	CW	4.00	N.A.	72.00	598.00	128.00	21	1.60	.09	.43	6.80	3.53	-N.A.-	-N.A.-
16	9/13/69	2130	CW	2.00	N.A.	53.00	409.00	217.00	53	-N.A.-	-N.A.-	-N.A.-	-N.A.-	-N.A.-	-N.A.-	-N.A.-
17	10/ 2/69	715	CW	3.00	N.A.	22.00	368.00	222.00	60	13.74	.03	.54	26.12	17.53	-N.A.-	-N.A.-

SAMPLE DATA

TABLE 25

DES MOINES, IOWA, STORM WATER POLLUTION CONTROL STUDY
STATION U-1 (W-1) W. SIDE INTERCEPTOR AT SCOTT

FWPCA CONTRACT NO 14-12-402

ITEM	DATE	TIME	TYPE	DUR HOUR	FLOW MGD	BOD MG/L LBS	SUSPENDED SOLIDS TOTAL MG/L LBS	SUSPENDED SOLIDS VOLATILE MG/L LBS	SUSPENDED SOLIDS VOLATILE PER CENT	NITROGEN AMMONIA MG/L LBS	NITROGEN NITRITE MG/L LBS	NITRATE MG/L LBS	PHOSPHATES TOTAL MG/L LBS	PHOSPHATES SOLUABLE MG/L LBS	CHLORIDES MG/L LBS	CHROMIUM UG/L LBS
18	10/ 5/69	1740	SD	1.00	N.A.	138.00	150.00	65.00	43	-N.A.-	-N.A.-	-N.A.-	-N.A.-	-N.A.-	-N.A.-	-N.A.-
19	10/ 5/69	1840	SD	1.00	N.A.	-N.A.-	-N.A.-	-N.A.-		10.57	.03	-N.A.-	31.00	20.81	-N.A.-	-N.A.-
20	10/ 5/69	2240	SD	1.00	N.A.	109.00	-N.A.-	-N.A.-		-N.A.-	-N.A.-	-N.A.-	-N.A.-	-N.A.-	-N.A.-	-N.A.-
21	10/ 5/69	2340	SD	1.00	N.A.	242.00	-N.A.-	-N.A.-		-N.A.-	-N.A.-	-N.A.-	-N.A.-	-N.A.-	-N.A.-	-N.A.-
22	10/ 6/69	040	SD	1.00	N.A.	-N.A.-	-N.A.-	-N.A.-		5.43	.07	.41	15.26	8.23	-N.A.-	-N.A.-
23	10/ 6/69	140	SD	1.00	N.A.	266.00	-N.A.-	-N.A.-		4.65	.06	.36	-N.A.-	-N.A.-	-N.A.-	-N.A.-

TABLE 25 CON'T

DES MOINES, IOWA, STORM WATER POLLUTION CONTROL STUDY
STATION U-1A (O-8A) INGERSOLL RUN SEWER AT 22ND AND HIGH

FWPCA CONTRACT NO 14-12-402

							SUSPENDED SOLIDS			NITROGEN			PHOSPHATES		CHLORIDES	CHROMIUM
ITEM	DATE	* TIME	TYPE	DUR HOUR	FLOW MGD	BOD MG/L LBS	TOTAL MG/L LBS	VOLATILE MG/L LBS	PER CENT	AMMONIA MG/L LBS	NITRITE MG/L LBS	NITRATE MG/L LBS	TOTAL MG/L LBS	SOLUABLE MG/L LBS	MG/L LBS	UG/L LBS
1	10/13/68	8 0	CD	24.00	.9	-N.A.-	-N.A.-	-N.A.-		-N.A.-	-N.A.-	-N.A.-	-N.A.-	-N.A.-	-N.A.-	-N.A.-
2	10/14/68	8 0	CD	24.00	1.1	-N.A.-	-N.A.-	-N.A.-		-N.A.-	-N.A.-	-N.A.-	-N.A.-	-N.A.-	-N.A.-	-N.A.-
3	10/15/68	8 0	CD	24.00	1.3	-N.A.-	-N.A.-	-N.A.-		-N.A.-	-N.A.-	-N.A.-	-N.A.-	-N.A.-	-N.A.-	-N.A.-
4	12/12/68	23 0	SD	1.00	N.A.	221.00	-N.A.-	-N.A.-		33.00	.35	4.07	28.60	28.00	-N.A.-	-N.A.-
5	12/13/68	9 0	SD	1.00	N.A.	221.00	-N.A.-	-N.A.-		35.80	.61	6.65	29.20	32.50	-N.A.-	-N.A.-
6	1/12/69	9 0	CD	24.00	1.2	228.00 2279.1	179.00 1789.3	168.00 1679.3	94	22.58 225.7	.10 1.0	3.04 30.4	17.25 172.4	9.84 98.4	-N.A.-	-N.A.-
7	1/12/69	9 0	CD	24.00	.9	197.00 1476.9	235.00 1761.8	218.00 1634.3	93	26.66 199.9	.08 .6	0.00 0.0	21.05 157.8	11.63 87.2	-N.A.-	-N.A.-
8	1/13/69	9 0	CD	24.00	.9	178.00 1334.5	224.00 1679.3	171.00 1282.0	76	26.11 195.7	0.00 0.0	0.00 0.0	18.32 137.3	11.50 86.2	-N.A.-	-N.A.-
9	7/18/69	1 0	SW	1.00	N.A.	58.00	90.00	70.00	78	-N.A.-	-N.A.-	-N.A.-	-N.A.-	-N.A.-	-N.A.-	-N.A.-
10	7/18/69	2 0	SW	1.00	N.A.	55.00	666.00	107.00	16	-N.A.-	-N.A.-	-N.A.-	-N.A.-	-N.A.-	-N.A.-	-N.A.-
11	7/18/69	3 0	SW	1.00	N.A.	38.00	-N.A.-	-N.A.-		.75	.04	.53	-N.A.-	.47	-N.A.-	-N.A.-
12	7/18/69	5 0	SW	1.00	N.A.	36.00	-N.A.-	-N.A.-		1.26	.05	1.41	-N.A.-	.81	-N.A.-	-N.A.-
13	8/ 6/69	1545	CW	3.00	N.A.	44.00	62.00	31.00	50	8.40	.01	.05	-N.A.-	19.50	-N.A.-	-N.A.-
14	8/ 7/69	545	CW	3.00	N.A.	154.00	303.00	101.00	33	7.28	.22	.57	-N.A.-	3.62	-N.A.-	-N.A.-
15	8/21/69	9 0	CW	2.00	N.A.	-N.A.-	394.00	195.00	49	6.55	.05	.27	-N.A.-	5.84	-N.A.-	-N.A.-

TABLE 25 CON'T

DES MOINES, IOWA, STORM WATER POLLUTION CONTROL STUDY
STATION D-2 (W-2) SOUTHWEST OUTFALL SEWER

FWPCA CONTRACT NO 14-12-402

							SUSPENDED SOLIDS			NITROGEN			PHOSPHATES		CHLORIDES	CHROMIUM
ITEM	DATE	TIME	TYPE	DUR HOUR	FLOW MGD	BOD MG/L LBS	TOTAL MG/L LBS	VOLATILE MG/L LBS	PER CENT	AMMONIA MG/L LBS	NITRITE MG/L LBS	NITRATE MG/L LBS	TOTAL MG/L LBS	SOLUABLE MG/L LBS	MG/L LBS	UG/L LBS
1	10/27/68	8 0	CD	24.00	4.2	-N.A.-	-N.A.-	-N.A.-		-N.A.-	-N.A.-	-N.A.-	-N.A.-	-N.A.-	-N.A.-	-N.A.-
2	10/28/68	8 0	CD	24.00	4.4	-N.A.-	-N.A.-	-N.A.-		-N.A.-	-N.A.-	-N.A.-	-N.A.-	-N.A.-	-N.A.-	-N.A.-
3	10/28/68	8 0	CD	24.00	4.0	-N.A.-	-N.A.-	-N.A.-		-N.A.-	-N.A.-	-N.A.-	-N.A.-	-N.A.-	-N.A.-	-N.A.-
4	1/12/69	10 0	CD	24.00	5.0	159.00 6622.3	185.00 7705.2	163.00 6788.9	80	22.30 928.8	.18 7.5	2.03 84.5	21.45 893.4	18.02 750.5	-N.A.-	-N.A.-
5	1/13/69	10 0	CD	24.00	5.3	200.00 8829.8	184.00 8123.4	169.00 7461.2	92	22.58 996.9	.07 3.1	0.00 0.0	25.30 1117.0	17.45 770.4	-N.A.-	-N.A.-
6	1/14/69	10 0	CD	24.00	5.1	180.00 7646.9	184.00 7816.9	167.00 7094.7	91	25.30 1074.8	0.00 0.0	0.00 0.0	19.36 822.5	14.45 613.9	-N.A.-	-N.A.-
7	9/15/69	10 0	CW	24.00	7.7	165.00 10583.3	185.00 11866.1	103.00 6606.5	56	14.04 900.5	.03 1.7	.21 13.5	37.60 2411.7	33.20 2129.5	-N.A.-	-N.A.-
8	9/22/69	13 0	SD	1.00	N.A.	202.00	-N.A.-	-N.A.-		-N.A.-	-N.A.-	-N.A.-	-N.A.-	-N.A.-	-N.A.-	-N.A.-
9	9/22/69	14 0	SW	1.00	N.A.	130.00	-N.A.-	-N.A.-		12.81	-N.A.-	.36	-N.A.-	2.75	-N.A.-	-N.A.-
10	9/22/69	16 0	SW	1.00	N.A.	56.00	107.00	63.00	59	-N.A.-	-N.A.-	-N.A.-	-N.A.-	-N.A.-	-N.A.-	-N.A.-
11	9/22/69	17 0	CW	3.00	N.A.	192.00	572.00	240.00	42	6.52	.44	.13	46.70	14.00	-N.A.-	-N.A.-
12	9/22/69	23 0	CW	12.00	N.A.	123.00	156.00	93.00	60	7.23	.11	.52	19.90	19.60	-N.A.-	-N.A.-

TABLE 25 CON'T

DES MOINES.IOWA.STORM WATER POLLUTION CONTROL STUDY
STATION D-3 (W-3) EAST SIDE INTERCEPTOR AT EAST 1ST AND RACCOON

FWPCA CONTRACT NO 14-12-402

ITEM	DATE	TIME	TYPE	DUR HOUR	FLOW MGD	BOD MG/L LBS	SUSPENDED SOLIDS TOTAL MG/L LBS	SUSPENDED SOLIDS VOLATILE MG/L LBS	SUSPENDED SOLIDS VOLATILE PER CENT	NITROGEN AMMONIA MG/L LBS	NITROGEN NITRITE MG/L LBS	NITRATE MG/L LBS	PHOSPHATES TOTAL MG/L LBS	PHOSPHATES SOLUABLE MG/L LBS	CHLORIDES MG/L LBS	CHROMIU UG/L LBS
1	10/27/68	8 0	CD	24.00	1.3	170.00 1940.9	-N.A.-	-N.A.-		20.50 222.0	7.68 83.2	27.60 298.9	38.60 418.0	39.20 424.5	-N.A.-	-N.A.-
2	10/28/68	8 0	CD	24.00	1.5	214.00 2673.9	-N.A.-	-N.A.-		19.80 247.4	.01 .1	.25 3.1	52.70 658.5	24.20 302.4	-N.A.-	-N.A.-
3	10/29/68	8 0	CD	24.00	1.6	-N.A.-	-N.A.-	-N.A.-		19.10 254.6	*TRACE*	.18 2.4	44.40 591.8	21.00 279.9	-N.A.-	-N.A.-
4	12/12/68	23 0	SD	1.00	N.A.	385.00	-N.A.-	-N.A.-		29.80	.13	1.09	43.50	42.20	-N.A.-	-N.A.-
5	12/13/68	9 0	SD	1.00	N.A.	206.00	-N.A.-	-N.A.-		31.90	.24	1.67	33.20	27.60	-N.A.-	-N.A.-
6	2/ 2/69	10 0	CD	24.00	1.7	104.00 1472.7	188.00 2662.3	154.00 2180.8	82	39.90 565.0	.09 1.3	0.00 0.0	5.14 72.8	2.79 39.5	-N.A.-	-N.A.-
7	2/ 3/69	10 0	CD	24.00	1.9	168.00 2658.9	170.00 2690.6	153.00 2421.5	90	23.80 376.7	.04 .6	0.00 0.0	4.99 79.0	2.43 38.5	-N.A.-	-N.A.-
8	2/ 4/69	1030	CW	2.00	2.4	-N.A.-	101.00 168.3	101.00 168.3	100	-N.A.-	-N.A.-	-N.A.-	-N.A.-	-N.A.-	109.00 181.6	-N.A.-
9	2/ 4/69	1230	SW	1.00	2.4	-N.A.-	162.00 134.9	134.00 111.6	83	-N.A.-	-N.A.-	-N.A.-	-N.A.-	-N.A.-	189.00 157.4	-N.A.-
10	2/ 4/69	1330	SW	1.00	4.0	-N.A.-	204.00 283.2	124.00 172.2	61	-N.A.-	-N.A.-	-N.A.-	-N.A.-	-N.A.-	817.00 1134.3	-N.A.-
11	2/ 4/69	1930	SW	1.00	2.6	-N.A.-	277.00 250.0	130.00 117.3	47	-N.A.-	-N.A.-	-N.A.-	-N.A.-	-N.A.-	595.00 536.9	-N.A.-
12	2/ 5/69	930	SW	1.00	2.3	-N.A.-	94.00 75.0	89.00 71.0	95	-N.A.-	-N.A.-	-N.A.-	-N.A.-	-N.A.-	95.00 75.8	-N.A.-
13	2/ 6/69	10 0	CD	24.00	2.1	-N.A.-	156.00 2728.9	137.00 2396.5	88	-N.A.-	-N.A.-	-N.A.-	-N.A.-	-N.A.-	-N.A.-	-N.A.-
14	6/25/69	11 0	CW	21.00	3.5	112.00 2857.2	174.00 4438.8	125.00 3188.8	72	15.80 403.1	.02 .6	.14 3.5	27.40 699.0	21.10 538.3	-N.A.-	-N.A.-
15	6/26/69	8 0	CW	2.00	10.3	120.00 458.0	491.00 3510.6	178.00 1272.7	36	-N.A.-	-N.A.-	-N.A.-	-N.A.-	-N.A.-	-N.A.-	-N.A.-
16	6/26/69	10 0	SW	1.00	7.9	-N.A.-	-N.A.-	-N.A.-		5.49 15.1	.14 .4	.86 2.4	-N.A.-	4.20 11.5	-N.A.-	-N.A.-
17	8/19/69	11 0	CW	23.00	3.3	114.00 3003.2	98.00 2581.7	61.00 1607.0	62	34.19 900.7	.02 .6	.07 1.8	24.26 639.1	18.20 479.5	-N.A.-	-N.A.-

TABLE 25 CON'T

DES MOINES.IOWA.STORM WATER POLLUTION CONTROL STUDY FWPCA CONTRACT NO 14-12-402
STATION D-3 (W-3) EAST SIDE INTERCEPTOR AT EAST 1ST AND RACCOON

ITEM	DATE	TIME	TYPE	DUR HOUR	FLOW MGD	BOD MG/L LBS	SUSPENDED SOLIDS TOTAL MG/L LBS	SUSPENDED SOLIDS VOLATILE MG/L LBS	SUSPENDED SOLIDS VOLATILE PER CENT	NITROGEN AMMONIA MG/L LBS	NITROGEN NITRITE MG/L LBS	NITRATE MG/L LBS	PHOSPHATES TOTAL MG/L LBS	PHOSPHATES SOLUABLE MG/L LBS	CHLORIDES MG/L LBS	CHROMIUM UG/L LBS
18	8/20/69	10 0	CW	4.00	8.6	148.00 1767.1	290.00 3462.5	219.00 2614.8	76	55.29 660.1	.12 1.4	.31 3.7	15.27 182.3	8.11 96.8	-N.A.-	-N.A.-
19	9/22/69	13 0	CB	5.00	8.2	147.00 2091.9	345.00 4909.5	173.00 2461.9	50	7.23 102.9	.24 3.4	.27 3.8	14.10 200.6	12.50 177.9	-N.A.-	-N.A.-
20	9/22/69	18 0	CW	2.00	24.1	97.00 1622.8	420.00 7026.4	129.00 2158.1	31	-N.A.-	-N.A.-	.34 5.7	-N.A.-	1.54 25.8	-N.A.-	-N.A.-
21	9/22/69	20 0	CD	15.00	2.9	123.00 1857.1	131.00 1977.9	74.00 1117.3	56	5.38 81.2	.44 6.6	.19 2.9	12.40 187.2	.87 13.1	-N.A.-	-N.A.-

TABLE 25 CON'T

DES MOINES-IOWA-STORM WATER POLLUTION CONTROL STUDY
STATION D-5 (W-5) SOUTH SIDE TRUNK

FWPCA CONTRACT NO 14-12-402

ITEM	DATE	TIME	TYPE	DUR HOUR	FLOW MGD	BOD MG/L LBS	SUSPENDED SOLIDS TOTAL MG/L LBS	SUSPENDED SOLIDS VOLATILE MG/L LBS	SUSPENDED SOLIDS VOLATILE PER CENT	NITROGEN AMMONIA MG/L LBS	NITROGEN NITRITE MG/L LBS	NITROGEN NITRATE MG/L LBS	PHOSPHATES TOTAL MG/L LBS	PHOSPHATES SOLUABLE MG/L LBS	CHLORIDES MG/L LBS	CHROMIUM UG/L LBS
1	11/10/68	10 0	CD	24.00	1.3	255.00 2761.4	-N.A.-	-N.A.-		15.45 167.3	.54 6.3	3.19 34.5	-N.A.-	-N.A.-	-N.A.-	-N.A.-
2	11/11/68	10 0	CD	24.00	1.6	206.00 2745.6	-N.A.-	-N.A.-		18.54 247.1	.02 .3	.70 9.3	-N.A.-	-N.A.-	-N.A.-	-N.A.-
3	11/12/68	10 0	CD	24.00	1.2	-N.A.-	-N.A.-	-N.A.-		28.10 280.9	.27 2.7	2.54 25.4	-N.A.-	-N.A.-	-N.A.-	-N.A.-
4	2/ 2/69	9 0	CD	24.00	1.2	245.00 2449.0	194.00 1939.2	165.00 1649.3	85	47.60 475.8	.08 .8	0.00 0.0	6.71 67.1	3.16 31.6	-N.A.-	-N.A.-
5	2/ 3/69	9 0	CD	24.00	1.1	174.00 1594.4	137.00 1255.3	125.00 1145.4	91	29.60 271.2	.20 1.8	0.00 0.0	6.50 59.6	2.87 26.3	-N.A.-	-N.A.-
6	2/ 4/69	1130	SW	1.00	N.A.	-N.A.-	303.00	254.00	84	-N.A.-	-N.A.-	-N.A.-	-N.A.-	-N.A.-	115.00	-N.A.-
7	2/ 4/69	1230	SW	1.00	N.A.	-N.A.-	271.00	239.00	88	-N.A.-	-N.A.-	-N.A.-	-N.A.-	-N.A.-	139.00	-N.A.-
8	2/ 4/69	1330	SW	1.00	N.A.	-N.A.-	180.00	137.00	76	-N.A.-	-N.A.-	-N.A.-	-N.A.-	-N.A.-	321.00	-N.A.-
9	2/ 4/69	1430	SW	1.00	N.A.	-N.A.-	1052.00	536.00	51	-N.A.-	-N.A.-	-N.A.-	-N.A.-	-N.A.-	717.00	-N.A.-
10	2/ 4/69	1530	SW	1.00	N.A.	-N.A.-	1037.00	548.00	53	-N.A.-	-N.A.-	-N.A.-	-N.A.-	-N.A.-	866.00	-N.A.-
11	2/ 4/69	1630	SW	1.00	N.A.	-N.A.-	849.00	458.00	54	-N.A.-	-N.A.-	-N.A.-	-N.A.-	-N.A.-	762.00	-N.A.-
12	2/ 4/69	1730	SW	1.00	N.A.	-N.A.-	429.00	244.00	57	-N.A.-	-N.A.-	-N.A.-	-N.A.-	-N.A.-	509.00	-N.A.-
13	2/ 4/69	1830	SW	1.00	N.A.	-N.A.-	377.00	275.00	73	-N.A.-	-N.A.-	-N.A.-	-N.A.-	-N.A.-	410.00	-N.A.-
14	2/ 5/69	430	SW	1.00	N.A.	-N.A.-	165.00	163.00	99	-N.A.-	-N.A.-	-N.A.-	-N.A.-	-N.A.-	298.00	-N.A.-
15	6/25/69	11 0	CW	4.00	N.A.	201.00	540.00	462.00	86	16.20	.31	.16	39.90	33.60	-N.A.-	-N.A.-
16	6/26/69	2 0	SW	1.00	N.A.	120.00	-N.A.-	-N.A.-		-N.A.-	-N.A.-	-N.A.-	-N.A.-	-N.A.-	-N.A.-	-N.A.-
17	6/26/69	8 0	CW	3.00	N.A.	218.00	966.00	434.00	45	14.20	.01	.06	16.50	6.77	-N.A.-	-N.A.-

TABLE 25 CON'T

DES MOINES,IOWA,STORM WATER POLLUTION CONTROL STUDY FWPCA CONTRACT NO 14-12-402
STATION D-5 (W-5) SOUTH SIDE TRUNK

							SUSPENDED SOLIDS			NITROGEN			PHOSPHATES		CHLORIDES	CHROMIUM
						.BOD	TOTAL	VOLATILE		AMMONIA	NITRITE	NITRATE	TOTAL	SOLUABLE		
ITEM	DATE	TIME	TYPE	DUR HOUR	FLOW MGD	MG/L LBS	MG/L LBS	MG/L LBS	PER CENT	MG/L LBS	MG/L LBS	MG/L LBS	MG/L LBS	MG/L LBS	MG/L LHS	UG/L LHS
18	6/26/69	18 0	SW	1.00	N.A.	122.00	1632.00	516.00	28	2.75	.02	.07	15.80	7.39	-N.A.-	-N.A.-
19	6/26/69	19 0	CW	2.00	N.A.	94.00	440.00	148.00	34	-N.A.-	-N.A.-	-N.A.-	-N.A.-	-N.A.-	-N.A.-	-N.A.-

TABLE 25 CON'T

DES MOINES.IOWA.STORM WATER POLLUTION CONTROL STUDY
STATION D-9 FOUR MILE TRUNK SEWER

FWPCA CONTRACT NO 14-12-402

							SUSPENDED SOLIDS			NITROGEN			PHOSPHATES		CHLORIDES	CHROMIUM
							TOTAL	VOLATILE		AMMONIA	NITRITE	NITRATE	TOTAL	SOLUABLE		
ITEM	DATE	TIME	TYPE	DUR HOUR	FLOW MGD	BOD MG/L LBS	MG/L LBS	MG/L LBS	PER CENT	MG/L LBS	MG/L LBS	MG/L LBS	MG/L LBS	MG/L LBS	MG/L LBS	UG/L LBS
1	2/ 2/69	10 0	CD	24.00	2.5	356.00 7413.7	189.00 3935.9	162.00 3373.6	86	25.75 536.2	.03 .6	0.00 0.0	5.56 115.8	3.70 77.1	-N.A.-	-N.A.-
2	2/ 3/69	10 0	CD	24.00	2.6	511.00 11067.2	178.00 3855.1	162.00 3508.6	91	26.00 563.1	.18 3.9	0.00 0.0	8.43 182.6	5.22 113.1	-N.A.-	-N.A.-
3	2/ 4/69	10 0	CD	24.00	3.1	181.00 4674.8	189.00 4880.5	161.00 4157.5	85	41.30 1066.5	.07 1.8	0.00 0.0	5.90 152.4	3.09 79.8	-N.A.-	-N.A.-

TABLE 25 CON'T

DES MOINES.IOWA.STORM WATER POLLUTION CONTROL STUDY
STATION U-2 CLOSES CREEK

FWPCA CONTRACT NO 14-12-402

ITEM	DATE	TIME	TYPE	DUR HOUR	FLOW CFS	BOD MG/L LBS	SUSPENDED SOLIDS TOTAL MG/L LBS	SUSPENDED SOLIDS VOLATILE MG/L LBS	SUSPENDED SOLIDS VOLATILE PER CENT	NITROGEN AMMONIA MG/L LBS	NITROGEN NITRITE MG/L LBS	NITRATE MG/L LBS	PHOSPHATES TOTAL MG/L LBS	PHOSPHATES SOLUABLE MG/L LBS	CHLORIDES MG/L LBS	CHROMIUM UG/L LBS
1	2/25/69	13 0	CS	12.00	14.1	144.00 6249.7	374.00 16448.8	94.00 4079.6	25	3.19 138.4	.02 .7	1.14 49.5	.27 11.7	.14 6.1	88.00 3819.2	33.00 1.4
2	2/26/69	16 0	CS	6.00	14.7	90.00 1783.2	861.00 17059.2	167.00 3308.8	19	3.07 60.8	.01 .2	1.01 20.0	.32 6.3	.27 5.3	59.00 1169.0	50.00 1.0
3	3/ 1/69	1430	SS	1.00	2.0	-N.A.-	-N.A.-	-N.A.-		-N.A.-	-N.A.-	-N.A.-	-N.A.-	-N.A.-	571.00 256.5	126.00 .1
4	3/ 1/69	1430	CS	8.00	5.4	39.00 378.5	716.00 6948.4	205.00 1989.4	29	1.41 13.7	.01 .1	.75 7.3	.16 1.6	.04 .4	-N.A.-	54.00 .5
5	3/ 2/69	1330	CS	11.00	2.6	35.00 224.9	236.00 1516.2	77.00 494.7	33	1.55 10.0	.01 .1	.58 3.7	.08 .5	.03 .2	-N.A.-	0.00 0.0
6	4/ 8/69	2230	CW	8.00	2.8	-N.A.-	862.00 4337.5	195.00 981.2	23	.91 4.6	.01 .1	1.51 7.6	1.55 7.8	.97 4.9	-N.A.-	-N.A.-
7	4/15/69	11 0	CR	12.00	5.8	18.00 281.4	310.00 4846.8	83.00 1297.7	27	.79 12.4	.00 .0	.90 14.1	-N.A.-	1.06 16.6	-N.A.-	-N.A.-
8	5/17/69	030	CR	16.00	17.2	20.00 1236.4	681.00 42100.1	115.00 7109.4	17	.55 34.0	.03 1.7	.56 34.6	3.36 207.7	1.64 101.4	-N.A.-	-N.A.-
9	5/19/69	15 0	SR	1.00	23.5	39.00 205.9	446.00 2354.5	96.00 506.8	22	.94 5.0	.14 .4	1.11 5.9	6.41 33.8	4.80 25.3	-N.A.-	-N.A.-
10	5/21/69	930	SR	.25	.2	19.00 .2	-N.A.-	-N.A.-		-N.A.-	-N.A.-	-N.A.-	-N.A.-	-N.A.-	-N.A.-	-N.A.-
11	5/21/69	930	CR	1.00	.2	16.00 .7	1167.00 52.4	180.00 8.1	15	1.18 .1	.04 .0	.92 .0	5.34 .2	4.14 .2	-N.A.-	-N.A.-
12	5/21/69	13 0	SR	1.00	210.0	17.00 802.0	-N.A.-	-N.A.-		-N.A.-	-N.A.-	-N.A.-	-N.A.-	-N.A.-	-N.A.-	-N.A.-
13	5/21/69	15 0	CR	2.00	50.5	-N.A.-	421.00 9551.9	74.00 1679.0	18	.78 17.7	.00 .1	1.40 31.8	3.69 83.7	2.50 56.7	-N.A.-	-N.A.-
14	5/21/69	20 0	SR	1.00	N.A.	24.00	150.00	62.00	41	-N.A.-	-N.A.-	-N.A.-	-N.A.-	-N.A.-	-N.A.-	-N.A.-
15	5/22/69	6 0	SR	1.00	N.A.	9.00	39.00	17.00	44	-N.A.-	-N.A.-	-N.A.-	-N.A.-	-N.A.-	-N.A.-	-N.A.-
16	7/ 9/69	5 0	CR	6.00	27.2	36.00 1319.4	970.00 35561.4	117.00 4289.4	12	.65 23.8	.02 .8	1.21 44.4	2.44 89.5	.53 19.4	-N.A.-	-N.A.-
17	9/22/69	1330	SR	1.00	24.5	22.00 171.1	508.00 2795.9	49.00 269.7	10	.51 2.8	.02 .1	.89 4.9	-N.A.-	.44 2.4	-N.A.-	-N.A.-

TABLE 25 CON'T

DES MOINES•IOWA•STORM WATER POLLUTION CONTROL STUDY
STATION U-2 CLOSES CREEK

FWPCA CONTRACT NO 14-12-402

ITEM	DATE	TIME	TYPE	DUR HOUR	FLOW CFS	BOD MG/L LBS	SUSPENDED SOLIDS TOTAL MG/L LBS	SUSPENDED SOLIDS VOLATILE MG/L LBS	SUSPENDED SOLIDS VOLATILE PER CENT	NITROGEN AMMONIA MG/L LBS	NITROGEN NITRITE MG/L LBS	NITRATE MG/L LBS	PHOSPHATES TOTAL MG/L LBS	PHOSPHATES SOLUABLE MG/L LBS	CHLORIDES MG/L LBS	CHROMIUM UG/L LBS
18	9/22/69	1430	SW	1.00	2.5	20.00 11.2	470.00 264.0	54.00 30.3	11	.46 .3	.02 .0	.84 .5	-N.A.-	.68 .4	-N.A.-	-N.A.-
19	9/22/69	1430	CR	5.00	25.3	33.00 937.8	611.00 17362.8	88.00 2500.7	14	.82 23.3	.04 1.2	1.04 29.6	1.85 52.6	1.08 30.7	-N.A.-	-N.A.-
20	9/22/69	2130	CR	6.00	3.7	205.00 1022.3	49.00 244.4	10.00 49.9	20	1.26 6.3	.08 .4	.47 2.3	.80 4.0	.80 4.0	-N.A.-	-N.A.-
21	10/ 5/69	1015	CR	6.00	3.9	20.00 105.1	154.00 809.5	60.00 315.4	39	.56 2.9	.01 .0	.36 1.9	1.67 8.8	.88 4.6	-N.A.-	-N.A.-
22	10/ 5/69	2215	CR	5.00	13.9	21.00 327.9	380.00 5932.7	60.00 936.7	16	1.87 29.2	.01 .1	.20 3.1	2.92 45.6	1.28 20.0	-N.A.-	-N.A.-

TABLE 25 CON'T

DES MOINES,IOWA,STORM WATER POLLUTION CONTROL STUDY
STATION U-3 WEST SIDE INTERCEPTOR AT PROSPECT AV MKMN

FWPCA CONTRACT NO 14-12-402

							SUSPENDED SOLIDS			NITROGEN			PHOSPHATES		CHLORIDES	CHROMIUM
ITEM	DATE	TIME	TYPE	DUR HOUR	FLOW CFS	BOD MG/L LBS	TOTAL MG/L LBS	VOLATILE MG/L LBS	PER CENT	AMMONIA MG/L LBS	NITRITE MG/L LBS	NITRATE MG/L LBS	TOTAL MG/L LBS	SOLUABLE MG/L LBS	MG/L LBS	UG/L LBS
1	7/ 9/69	945	SR	1.00	N.A.	62.00	105.00	60.00	56	-N.A.-	-N.A.-	-N.A.-	-N.A.-	-N.A.-	-N.A.-	-N.A.-
2	7/ 9/69	915	SR	1.00	N.A.	40.00	118.00	62.00	53	-N.A.-	-N.A.-	-N.A.-	-N.A.-	-N.A.-	-N.A.-	-N.A.-
3	7/17/69	9 0	CR	24.00	N.A.	106.00	205.00	110.00	54	4.53	.08	.26	11.72	8.25	-N.A.-	-N.A.-

TABLE 25 CON'T

DES MOINES.IOWA.STORM WATER POLLUTION CONTROL STUDY
STATION U-6 WEST SIDE INTERCEPTOR AT GRAND AVE.

FWPCA CONTRACT NO 14-12-402

							SUSPENDED SOLIDS			NITROGEN			PHOSPHATES		CHLORIDES	CHROMIUM
ITEM	DATE	TIME	TYPE	DUR HOUR	FLOW CFS	BOD MG/L LBS	TOTAL MG/L LBS	VOLATILE MG/L LBS	PER CENT	AMMONIA MG/L LBS	NITRITE MG/L LBS	NITRATE MG/L LBS	TOTAL MG/L LBS	SOLUABLE MG/L LBS	MG/L LBS	UG/L LBS
1	5/21/69	1015	CR	1.50	N.A.	44.00	889.00	201.00	23	3.98	.06	.60	9.38	6.53	-N.A.-	-N.A.-
2	5/21/69	1130	CR	.50	N.A.	69.00	943.00	179.00	21	-N.A.-	-N.A.-	-N.A.-	-N.A.-	-N.A.-	-N.A.-	-N.A.-
3	5/21/69	12 0	CR	.50	25.0	29.00 81.4	1166.00 3274.1	165.00 463.3	14	-N.A.-	-N.A.-	-N.A.-	-N.A.-	-N.A.-	-N.A.-	-N.A.-
4	5/21/69	1230	CR	.50	N.A.	-N.A.-	-N.A.-	-N.A.-		.55	.02	.76	4.31	2.45	-N.A.-	-N.A.-
5	5/21/69	1330	CR	1.25	N.A.	34.00	745.00	152.00	20	.50	.08	1.32	5.52	3.05	-N.A.-	-N.A.-
6	5/21/69	16 0	CR	.50	N.A.	56.00	237.00	113.00	48	-N.A.-	-N.A.-	-N.A.-	-N.A.-	-N.A.-	-N.A.-	-N.A.-
7	5/17/69	930	CR	2.00	N.A.	118.00	-N.A.-	-N.A.-		6.88	.04	-N.A.-	-N.A.-	-N.A.-	-N.A.-	-N.A.-
8	8/ 6/69	1530	CR	19.00	13.8	76.00 4476.4	155.00 9129.6	102.00 6007.9	66	13.20 777.5	.45 26.5	.70 41.2	-N.A.-	258.00 15196.4	-N.A.-	-N.A.-
9	8/ 7/69	530	SR	1.00	11.6	215.00 560.3	759.00 1977.8	434.00 1130.9	57	-N.A.-	-N.A.-	-N.A.-	-N.A.-	-N.A.-	-N.A.-	-N.A.-
10	8/21/69	9 0	SR	2.00	12.8	150.00 862.6	225.00 1293.9	114.00 655.6	51	27.40 157.6	.07 .4	.20 1.2	-N.A.-	11.40 65.6	-N.A.-	-N.A.-
11	9/22/69	1240	CR	22.00	15.9	158.00 12415.5	308.00 24202.4	167.00 13122.7	54	13.43 1055.3	.27 21.2	.18 14.1	20.30 1595.2	22.80 1791.6	-N.A.-	-N.A.-

TABLE 25 CON'T

DES MOINES, IOWA, STORM WATER POLLUTION CONTROL STUDY
STATION O-7 WEST SIDE STORM BOX

FWPCA CONTRACT NO 14-12-402

ITEM	DATE	TIME	TYPE	DUR HOUR	FLOW CFS	BOD MG/L LBS	SUSPENDED SOLIDS TOTAL MG/L LBS	SUSPENDED SOLIDS VOLATILE MG/L LBS	SUSPENDED SOLIDS VOLATILE PER CENT	NITROGEN AMMONIA MG/L LBS	NITROGEN NITRITE MG/L LBS	NITRATE MG/L LBS	PHOSPHATES TOTAL MG/L LBS	PHOSPHATES SOLUABLE MG/L LBS	CHLORIDES MG/L LBS	CHROMIUM UG/L LBS
1	2/ 5/69	1720	SS	1.00	N.A.	-N.A.-	-N.A.-	-N.A.-		-N.A.-	-N.A.-	-N.A.-	-N.A.-	-N.A.-	71.00	7.00
2	2/ 6/69	945	SS	1.00	N.A.	-N.A.-	-N.A.-	-N.A.-		-N.A.-	-N.A.-	-N.A.-	-N.A.-	-N.A.-	46.00	5.00
3	2/ 6/69	1445	SS	1.00	N.A.	-N.A.-	-N.A.-	-N.A.-		-N.A.-	-N.A.-	-N.A.-	-N.A.-	-N.A.-	57.00	4.00
4	10/ 2/69	430	CR	4.00	N.A.	42.00	70.00	20.00	29	1.16	.01	3.09	1.30	1.29	-N.A.-	-N.A.-
5	10/ 2/69	830	CP	2.00	N.A.	109.00	140.00	44.00	31	2.38	.00	.15	5.37	2.62	-N.A.-	-N.A.-
6	10/ 2/69	1030	CR	2.00	N.A.	51.00	84.00	26.00	31	2.18	.01	3.97	2.50	2.06	-N.A.-	-N.A.-
7	10/ 5/69	1445	CR	3.00	N.A.	34.00	76.00	42.00	55	.32	.01	.10	1.46	.45	-N.A.-	-N.A.-
8	10/ 5/69	1745	SR	1.00	N.A.	41.00	364.00	109.00	30	-N.A.-	-N.A.-	-N.A.-	-N.A.-	-N.A.-	-N.A.-	-N.A.-
9	10/ 5/69	1845	CR	4.00	N.A.	53.00	60.00	16.00	27	3.44	.00	.24	11.10	6.38	-N.A.-	-N.A.-
10	10/ 5/69	2245	SR	1.00	N.A.	20.00	-N.A.-	-N.A.-		0.00	.01	.20	-N.A.-	-N.A.-	-N.A.-	-N.A.-
11	10/ 5/69	2345	SR	1.00	N.A.	67.00	506.00	104.00	20	-N.A.-	-N.A.-	-N.A.-	-N.A.-	-N.A.-	-N.A.-	-N.A.-
12	10/ 6/69	845	SR	1.00	N.A.	-N.A.-	-N.A.-	-N.A.-		4.41	.01	.46	20.18	6.67	-N.A.-	-N.A.-
13	10/ 6/69	145	CR	9.00	N.A.	30.00	170.00	71.00	42	.82	.01	.16	7.75	1.67	-N.A.-	-N.A.-

DES MOINES-IOWA-STORM WATER POLLUTION CONTROL STUDY
STATION 0-8 INGERSOLL RUN OVERFLOW AT OUTLET

FWPCA CONTRACT NO 14-12-402

							SUSPENDED SOLIDS			NITROGEN			PHOSPHATES		CHLORIDES	CHROMIUM
ITEM	DATE	TIME	TYPE	DUR HOUR	FLOW CFS	BOD MG/L LBS	TOTAL MG/L LBS	VOLATILE MG/L LBS	PER CENT	AMMONIA MG/L LBS	NITRITE MG/L LBS	NITRATE MG/L LBS	TOTAL MG/L LBS	SOLUABLE MG/L LBS	MG/L LBS	UG/L LBS
1	2/24/69	1555	SS	1.00	.9	31.00 6.3	627.00 126.8	194.00 39.2	31	4.99 1.0	.01 .0	.19 .0	-N.A.-	-N.A.-	150.00 30.3	-N.A.-
2	3/17/69	11 0	CR	8.00	8.5	61.00 931.8	229.00 3499.1	89.00 1359.5	39	11.22 171.4	.01 .1	0.00 0.0	-N.A.-	.05 .8	-N.A.-	-N.A.-
3	4/14/69	1030	CR	9.00	7.7	35.00 544.9	286.00 4452.3	114.00 1774.7	40	2.44 38.0	.01 .2	1.03 16.0	8.57 133.4	4.12 64.1	-N.A.-	-N.A.-
4	4/17/69	130	CR	16.00	6.8	32.00 782.1	150.00 3666.1	68.00 1662.0	45	2.42 59.1	.01 .1	1.21 29.6	3.77 92.1	2.61 63.8	-N.A.-	-N.A.-
5	4/26/69	1915	CR	4.00	37.5	38.00 1280.4	311.00 10479.5	218.00 7345.7	70	.85 28.6	.00 .1	.69 23.3	1.86 62.7	9.31 313.7	-N.A.-	-N.A.-
6	4/27/69	2315	CR	12.00	1.2	53.00 171.4	91.00 294.4	26.00 84.1	29	9.03 29.2	.05 .2	4.42 14.3	1.48 4.8	8.80 28.5	-N.A.-	-N.A.-
7	5/ 7/69	17 0	CR	6.00	37.8	45.00 2292.7	343.00 17475.3	76.00 3872.1	22	.01 .5	.00 .2	.45 22.9	-N.A.-	-N.A.-	-N.A.-	-N.A.-
8	5/ 7/69	1730	SR	.50	31.2	130.00 455.6	1060.00 3714.6	275.00 963.7	26	-N.A.-	-N.A.-	-N.A.-	-N.A.-	-N.A.-	-N.A.-	-N.A.-
9	5/ 7/69	18 0	SR	1.00	12.2	-N.A.-	-N.A.-	-N.A.-		2.10 5.8	.01 .0	.82 2.2	6.41 17.6	-N.A.-	-N.A.-	-N.A.-
10	5/ 7/69	19 0	SR	.75	47.5	61.00 488.2	493.00 3945.4	123.00 984.3	25	-N.A.-	-N.A.-	-N.A.-	-N.A.-	-N.A.-	-N.A.-	-N.A.-
11	5/ 7/69	1930	SR	.50	46.0	-N.A.-	-N.A.-	-N.A.-		.66 3.4	.00 .0	.70 3.6	2.99 15.4	-N.A.-	-N.A.-	-N.A.-
12	5/ 7/69	21 0	SR	.75	46.0	33.00 255.8	263.00 2038.3	60.00 465.0	23	-N.A.-	-N.A.-	-N.A.-	-N.A.-	-N.A.-	-N.A.-	-N.A.-
13	5/ 7/69	2130	SR	.50	46.0	-N.A.-	-N.A.-	-N.A.-		.40 2.1	.00 .0	.64 3.3	2.10 10.9	-N.A.-	-N.A.-	-N.A.-
14	5/16/69	2330	CR	6.00	11.0	96.00 1423.3	645.00 9562.9	212.00 3143.2	33	1.83 27.1	.04 .7	.60 8.4	7.33 108.7	3.91 58.0	-N.A.-	-N.A.-
15	5/21/69	9 6	CR	2.00	28.6	53.00 681.0	560.00 7195.7	141.00 1811.8	25	-N.A.-	-N.A.-	-N.A.-	-N.A.-	-N.A.-	-N.A.-	-N.A.-
16	5/21/69	1130	CR	2.00	173.5	18.00 1403.1	-N.A.-	-N.A.-		.13 10.1	.01 1.1	.62 48.3	-N.A.-	-N.A.-	-N.A.-	-N.A.-
17	5/21/69	1330	CR	2.00	67.9	26.00 793.2	107.00 3264.2	35.00 1067.7	33	-N.A.-	-N.A.-	-N.A.-	-N.A.-	-N.A.-	-N.A.-	-N.A.-

TABLE 25 CON'T

DES MOINES-IOWA-STORM WATER POLLUTION CONTROL STUDY
STATION O-P INGERSOLL RUN OVERFLOW AT OUTLET

FWPCA CONTRACT NO 14-12-402

							SUSPENDED SOLIDS			NITROGEN			PHOSPHATES		CHLORIDES	CHROMIUM
						BOD	TOTAL	VOLATILE		AMMONIA	NITRITE	NITRATE	TOTAL	SOLUABLE		
ITEM	DATE	TIME	TYPE	DUR HOUR	FLOW CFS	MG/L LBS	MG/L LBS	MG/L LBS	PER CENT	MG/L LBS	MG/L LBS	MG/L LBS	MG/L LBS	MG/L LBS	MG/L LBS	UG/L LBS
18	5/22/69	1530	CR	17.00	2.3	37.00	60.00	36.00		2.98	.08	4.44	7.62	3.93	-N.A.-	-N.A.-
						325.0	527.0	316.2	60	26.2	.7	39.0	66.9	34.5		
19	8/ 7/69	5 0	CR	4.00	6.1	105.00	380.00	145.00		2.81	.22	.84	-N.A.-	2.68	-N.A.-	-N.A.-
						575.5	2082.9	794.8	38	15.4	1.2	4.6		14.7		
20	9/22/69	1450	SR	.25	27.0	220.00	1549.00	582.00		-N.A.-	-N.A.-	-N.A.-	-N.A.-	-N.A.-	-N.A.-	-N.A.-
						333.6	2348.8	882.5	38							
21	9/22/69	15 0	CR	2.00	28.8	115.00	559.00	164.00		2.13	.05	.93	9.60	1.92	-N.A.-	-N.A.-
						1488.0	7233.0	2122.0	29	27.6	.7	12.0	124.2	24.8		
22	9/22/69	1715	CR	2.00	16.4	23.00	16.00	2.00		.39	.11	1.27	.83	.79	-N.A.-	-N.A.-
						169.5	117.9	14.7	13	2.9	.8	9.4	6.1	5.8		
23	10/ 5/69	17 0	SR	1.25	10.4	128.00	308.00	174.00		8.20	.00	.17	21.47	13.18	-N.A.-	-N.A.-
						373.8	899.5	508.1	56	23.9	.0	.5	62.7	38.5		
24	10/ 5/69	18 0	SR	1.25	2.3	82.00	154.00	101.00		5.34	.01	.24	16.46	9.72	-N.A.-	-N.A.-
						53.0	99.5	65.2	66	3.4	.0	.2	10.6	6.3		

TABLE 25 CON'T

DES MOINES,IOWA,STORM WATER POLLUTION CONTROL STUDY
STATION U-RA INGERSOLL RUN OVERFLOW AT 22ND AND HIGH

FWPLA CONTRACT NO 14-12-402

							SUSPENDED SOLIDS			NITROGEN			PHOSPHATES		CHLORIDES	CHROMIUM
						BOD	TOTAL	VOLATILE		AMMONIA	NITRITE	NITRATE	TOTAL	SOLUABLE		
ITEM	DATE	TIME	TYPE	DUR HOUR	FLOW CFS	MG/L LHS	MG/L LHS	MG/L LHS	PER CENT	MG/L LHS	MG/L LHS	MG/L LHS	MG/L LHS	MG/L LHS	MG/L LHS	UG/L LHS
1	7/18/69	1 0	SR	1.00	N.A.	58.00	90.00	70.00	78	-N.A.-	-N.A.-	-N.A.-	-N.A.-	-N.A.-	-N.A.-	-N.A.-
2	7/18/69	2 0	SR	1.00	N.A.	55.00	666.00	107.00	16	-N.A.-	-N.A.-	-N.A.-	-N.A.-	-N.A.-	-N.A.-	-N.A.-
3	7/18/69	3 0	SR	1.00	N.A.	38.00	-N.A.-	-N.A.-		.75	.04	.53	-N.A.-	.47	-N.A.-	-N.A.-
4	7/18/69	5 0	SR	1.00	N.A.	36.00	-N.A.-	-N.A.-		1.26	.05	1.41	-N.A.-	.81	-N.A.-	-N.A.-
5	8/ 6/69	1545	CR	3.00	N.A.	44.00	62.00	31.00	50	8.88	.01	.05	-N.A.-	19.50	-N.A.-	-N.A.-
6	8/ 7/69	545	CR	3.00	N.A.	157.00	303.00	101.00	33	7.28	.02	.57	-N.A.-	3.62	-N.A.-	-N.A.-
7	8/21/69	9 0	CR	2.00	N.A.	150.00	394.00	195.00	49	6.55	.05	.27	-N.A.-	5.84	-N.A.-	-N.A.-

TABLE 25 CON'T

DES MOINES•IOWA•STORM WATER POLLUTION CONTROL STUDY
STATION S-1 THOMPSON AVE. STORM SEWER

FWPCA CONTRACT NO 14-12-402

ITEM	DATE	TIME	TYPE	DUR HOUR	FLOW CFS	BOD MG/L LBS	SUSPENDED SOLIDS TOTAL MG/L LBS	SUSPENDED SOLIDS VOLATILE MG/L LBS	SUSPENDED SOLIDS VOLATILE PER CENT	NITROGEN AMMONIA MG/L LBS	NITROGEN NITRITE MG/L LBS	NITROGEN NITRATE MG/L LBS	PHOSPHATES TOTAL MG/L LBS	PHOSPHATES SOLUABLE MG/L LBS	CHLORIDES MG/L LBS	CHROMIUM UG/L LBS
1	1/15/69	17 0	CS	17.00	1.4	48.00 130.0	223.00 1532.9	92.00 632.4	41	-N.A.-	-N.A.-	-N.A.-	4.42 30.4	.02 .1	125.00 859.2	-N.A.-
2	1/15/69	17 0	CS	2.00	.4	103.00 18.5	624.00 112.1	187.00 33.6	30	-N.A.-	-N.A.-	-N.A.-	-N.A.-	-N.A.-	455.00 81.8	-N.A.-
3	1/15/69	23 0	CS	2.00	2.5	70.00 78.4	319.00 356.3	112.00 125.8	35	-N.A.-	-N.A.-	-N.A.-	-N.A.-	-N.A.-	121.00 135.9	-N.A.-
4	1/16/69	2 0	CS	2.00	3.4	32.00 48.9	93.00 142.1	44.00 67.2	47	-N.A.-	-N.A.-	-N.A.-	-N.A.-	-N.A.-	71.00 108.5	-N.A.-
5	1/16/69	8 0	CS	2.00	1.3	36.00 21.0	183.00 106.9	70.00 40.9	38	-N.A.-	-N.A.-	-N.A.-	-N.A.-	-N.A.-	75.00 43.8	-N.A.-
6	1/16/69	10 0	CS	18.00	2.4	37.00 359.1	261.00 2532.9	107.00 1038.4	41	-N.A.-	-N.A.-	-N.A.-	.24 2.3	.01 .1	32.00 310.5	-N.A.-
7	1/16/69	10 0	SS	1.00	1.4	-N.A.-	-N.A.-	-N.A.-		-N.A.-	-N.A.-	-N.A.-	-N.A.-	-N.A.-	82.00 25.8	-N.A.-
8	1/16/69	11 0	SS	1.00	2.6	46.00 26.9	-N.A.-	-N.A.-		-N.A.-	-N.A.-	-N.A.-	-N.A.-	-N.A.-	62.00 36.2	-N.A.-
9	1/16/69	12 0	SS	1.00	5.5	-N.A.-	661.00 816.7	165.00 203.9	25	-N.A.-	-N.A.-	-N.A.-	-N.A.-	-N.A.-	52.00 64.2	-N.A.-
10	1/16/69	13 0	SS	1.00	7.2	-N.A.-	-N.A.-	-N.A.-		-N.A.-	-N.A.-	-N.A.-	-N.A.-	-N.A.-	29.00 46.9	-N.A.-
11	1/16/69	14 0	SS	1.00	6.4	-N.A.-	-N.A.-	-N.A.-		-N.A.-	-N.A.-	-N.A.-	-N.A.-	-N.A.-	28.00 40.3	-N.A.-
12	1/16/69	15 0	SS	1.00	5.1	-N.A.-	-N.A.-	-N.A.-		-N.A.-	-N.A.-	-N.A.-	-N.A.-	-N.A.-	13.00 14.9	-N.A.-
13	1/16/69	17 0	SS	1.00	2.4	28.00 15.1	167.00 90.0	68.00 36.7	41	-N.A.-	-N.A.-	-N.A.-	-N.A.-	-N.A.-	12.00 6.5	-N.A.-
14	1/16/69	22 0	SS	1.00	.8	16.00 2.9	55.00 9.9	35.00 6.3	64	-N.A.-	-N.A.-	-N.A.-	-N.A.-	-N.A.-	10.00 1.8	-N.A.-
15	1/17/69	3 0	SS	1.00	.6	21.00 2.8	24.00 3.2	24.00 3.2	100	-N.A.-	-N.A.-	-N.A.-	-N.A.-	-N.A.-	9.00 1.2	-N.A.-
16	2/ 4/69	1415	SS	1.00	3.9	-N.A.-	-N.A.-	-N.A.-		-N.A.-	-N.A.-	-N.A.-	-N.A.-	-N.A.-	1477.00 1294.0	44.00 .0
17	2/ 4/69	1515	SS	1.00	3.7	-N.A.-	557.00 463.0	145.00 120.5	26	-N.A.-	-N.A.-	-N.A.-	-N.A.-	-N.A.-	1228.00 1020.7	-N.A.-

TABLE 25 CON'T

DES MOINES,IOWA,STORM WATER POLLUTION CONTROL STUDY
STATION S-1 THOMPSON AVE. STORM SEWER

FWPCA CONTRACT NO 14-12-402

ITEM	DATE	TIME	TYPE	DUR HOUR	FLOW CFS	BOD MG/L LBS	SUSPENDED SOLIDS TOTAL MG/L LBS	SUSPENDED SOLIDS VOLATILE MG/L LBS	SUSPENDED SOLIDS VOLATILE PER CENT	NITROGEN AMMONIA MG/L LBS	NITROGEN NITRITE MG/L LBS	NITROGEN NITRATE MG/L LBS	PHOSPHATES TOTAL MG/L LBS	PHOSPHATES SOLUABLE MG/L LBS	CHLORIDES MG/L LBS	CHROMIUM UG/L LBS
18	2/ 4/69	1615	SS	1.00	2.5	-N.A.-	-N.A.-	-N.A.-		-N.A.-	-N.A.-	-N.A.-	-N.A.-	-N.A.-	1045.00 586.9	37.00 .0
19	2/ 4/69	1715	SS	1.00	1.1	-N.A.-	318.00 78.6	86.00 21.3	27	-N.A.-	-N.A.-	-N.A.-	-N.A.-	-N.A.-	920.00 227.3	-N.A.-
20	2/ 5/69	1215	SS	1.00	.2	-N.A.-	-N.A.-	-N.A.-		-N.A.-	-N.A.-	-N.A.-	-N.A.-	-N.A.-	743.00 33.4	21.00 .0
21	2/ 5/69	1315	SS	1.00	1.5	-N.A.-	200.00 69.4	62.00 20.9	30	-N.A.-	-N.A.-	-N.A.-	-N.A.-	-N.A.-	260.00 87.6	-N.A.-
22	2/ 5/69	1515	SS	1.00	3.1	-N.A.-	-N.A.-	-N.A.-		-N.A.-	-N.A.-	-N.A.-	-N.A.-	-N.A.-	579.00 403.2	-N.A.-
23	2/ 5/69	1615	SS	1.00	2.5	-N.A.-	-N.A.-	-N.A.-		-N.A.-	-N.A.-	-N.A.-	-N.A.-	-N.A.-	502.00 281.9	-N.A.-
24	2/21/69	1440	SS	1.00	.4	-N.A.-	-N.A.-	-N.A.-		-N.A.-	-N.A.-	-N.A.-	-N.A.-	-N.A.-	2317.00 208.2	-N.A.-
25	2/21/69	16 0	SS	1.00	.9	-N.A.-	1106.00 223.6	254.00 51.4	23	-N.A.-	-N.A.-	-N.A.-	-N.A.-	-N.A.-	964.00 194.9	168.00 .0
26	2/22/69	1045	SS	1.00	.2	-N.A.-	-N.A.-	-N.A.-		-N.A.-	-N.A.-	-N.A.-	-N.A.-	-N.A.-	315.00 14.2	-0.00 0.0
27	2/22/69	16 5	SS	1.00	.8	-N.A.-	450.00 80.9	122.00 21.9	27	-N.A.-	-N.A.-	-N.A.-	-N.A.-	-N.A.-	248.00 44.6	218.00 .0
28	2/24/69	1050	SS	.50	.2	-N.A.-	33.00 .7	13.00 .3	39	1.08 .0	.14 .0	1.11 .0	-N.A.-	-N.A.-	137.00 3.1	-N.A.-
29	2/24/69	1515	SS	.50	2.5	-N.A.-	649.00 182.2	175.00 49.1	27	3.19 .9	.52 .1	1.32 .4	-N.A.-	-N.A.-	237.00 66.5	-N.A.-
30	2/24/69	18 0	SS	.50	1.4	-N.A.-	500.00 78.6	135.00 21.2	27	-N.A.-	-N.A.-	-N.A.-	-N.A.-	-N.A.-	-N.A.-	-N.A.-
31	2/24/69	21 0	SS	.50	.6	-N.A.-	-N.A.-	-N.A.-		-N.A.-	-N.A.-	-N.A.-	-N.A.-	-N.A.-	152.00 10.2	148.00 .0
32	2/24/69	1440	CS	7.00	1.5	70.00 165.1	471.00 1111.0	146.00 344.4	31	3.66 8.6	.24 .6	1.61 3.8	-N.A.-	.26 .6	-N.A.-	-N.A.-
33	2/25/69	10 0	CS	12.00	3.3	140.00 1245.4	522.00 4643.6	177.00 1574.5	34	.61 5.4	.16 1.4	1.04 9.3	.31 2.8	.10 .9	51.00 453.7	138.00 1.2
34	3/ 1/69	14 0	CS	6.00	1.9	18.00 97.1	662.00 1695.3	159.00 407.2	24	.38 1.0	.11 .3	.80 2.0	.10 .3	.02 .1	-N.A.-	41.00 .1

TABLE 25 CON'T

DES MOINES,IOWA,STORM WATER POLLUTION CONTROL STUDY
STATION S-1 THOMPSON AVE. STORM SEWER

FWPCA CONTRACT NO 14-12-402

							SUSPENDED SOLIDS			NITROGEN			PHOSPHATES		CHLORIDES	CHROMIUM
						BOD	TOTAL	VOLATILE		AMMONIA	NITRITE	NITRATE	TOTAL	SOLUABLE		
ITEM	DATE	TIME	TYPE	DUR HOUR	FLOW CFS	MG/L LBS	MG/L LBS	MG/L LBS	PER CENT	MG/L LBS	MG/L LBS	MG/L LBS	MG/L LBS	MG/L LBS	MG/L LBS	UG/L LBS
35	3/ 1/69	14 0	SS	1.00	2.4	-N.A.-	-N.A.-	-N.A.-		-N.A.-	-N.A.-	-N.A.-	-N.A.-	-N.A.-	226.00 121.8	-N.A.-
36	6/25/69	16 0	SR	1.00	.2	131.00 5.4	156.00 7.0	32.00 1.4	21	-N.A.-	-N.A.-	-N.A.-	-N.A.-	-N.A.-	-N.A.-	-N.A.-
37	6/26/69	7 0	SR	.50	.3	23.00 .8	50.00 1.7	-N.A.-		-N.A.-	-N.A.-	-N.A.-	-N.A.-	-N.A.-	-N.A.-	-N.A.-
38	6/28/69	4 0	SR	1.00	3.4	19.00 16.2	50.00 42.7	-N.A.-		-N.A.-	-N.A.-	-N.A.-	-N.A.-	-N.A.-	-N.A.-	-N.A.-
39	8/20/69	930	CR	4.50	2.2	38.00 84.5	138.00 306.9	45.00 100.1	33	4.36 9.7	.08 .2	.97 2.2	1.75 3.9	.54 1.2	-N.A.-	-N.A.-
40	9/ 4/69	845	SR	.25	1.9	40.00 8.8	220.00 48.2	68.00 14.9	31	-N.A.-	-N.A.-	-N.A.-	-N.A.-	-N.A.-	-N.A.-	-N.A.-
41	9/ 4/69	9 0	SR	.50	4.3	-N.A.-	-N.A.-	-N.A.-		1.94 .9	.04 .0	1.30 .6	.88 .4	.41 .2	-N.A.-	-N.A.-
42	9/ 4/69	10 0	SR	.50	2.4	19.00 5.1	129.00 34.8	40.00 10.8	31	-N.A.-	-N.A.-	-N.A.-	-N.A.-	-N.A.-	-N.A.-	-N.A.-
43	9/22/69	1540	SR	.50	1.8	44.00 18.8	305.00 130.2	80.00 34.1	26	-N.A.-	-N.A.-	-N.A.-	-N.A.-	-N.A.-	-N.A.-	-N.A.-
44	9/22/69	16 0	CR	1.50	4.2	30.00 42.5	294.00 416.1	75.00 106.1	26	1.07 1.5	.04 .1	.97 1.4	1.06 1.5	.01 .0	-N.A.-	-N.A.-
45	9/22/69	1745	CR	.75	9.9	20.00 33.4	464.00 773.9	142.00 236.8	31	-N.A.-	-N.A.-	-N.A.-	-N.A.-	-N.A.-	-N.A.-	-N.A.-
46	10/ 2/69	830	SR	1.00	0.0	50.00 0.0	-N.A.-	-N.A.-		-N.A.-	-N.A.-	-N.A.-	-N.A.-	-N.A.-	-N.A.-	-N.A.-
47	10/30/69	4 0	CR	8.00	.4	35.00 25.2	44.00 31.6	32.00 23.0	73	.79 .6	.06 .0	.87 .6	-N.A.-	.24 .2	-N.A.-	-N.A.-
48	10/30/69	10 0	SR	1.00	.4	56.00 5.0	145.00 13.0	69.00 6.2	48	-N.A.-	-N.A.-	-N.A.-	-N.A.-	-N.A.-	-N.A.-	-N.A.-

TABLE 25 CON'T

DES MOINES,IOWA,STORM WATER POLLUTION CONTROL STUDY
STATION S-3 CUMMINS PARKWAY STORM SEWER

FWPCA CONTRACT NO 14-12-402

ITEM	DATE	TIME	TYPE	COMP HOUR	FLOW CFS	BOD MG/L LBS	SUSPENDED SOLIDS TOTAL MG/L LBS	VOLATILE MG/L LBS	PER CENT	NITROGEN AMMONIA MG/L LBS	NITRITE MG/L LBS	NITRATE MG/L LBS	PHOSPHATES TOTAL MG/L LBS	SOLUABLF MG/L LBS	CHLORIDES MG/L LBS	CHROMIUM UG/L LBS
1	1/15/69	18 0	CS	24.00	1.8	31.00 300.8	302.00 2930.7	90.00 873.4	30	-N.A.-	-N.A.-	2.17 21.1	-N.A.-	.12 1.2	100.00 970.4	-N.A.-
2	1/15/69	19 0	SS	4.00	.3	64.00 17.3	450.00 121.3	140.00 37.7	31	-N.A.-	-N.A.-	-N.A.-	-N.A.-	-N.A.-	2004.00 540.2	1210.00 .3
3	1/15/69	22 0	SS	4.00	.9	-N.A.-	-N.A.-	-N.A.-		-N.A.-	-N.A.-	-N.A.-	-N.A.-	-N.A.-	1608.00 1300.4	-N.A.-
4	1/16/69	2 0	SS	4.00	3.3	24.00 71.2	454.00 1346.2	77.00 228.3	17	-N.A.-	-N.A.-	-N.A.-	-N.A.-	-N.A.-	367.00 1088.2	-N.A.-
5	1/16/69	6 0	SS	4.00	1.2	-N.A.-	-N.A.-	-N.A.-		-N.A.-	-N.A.-	-N.A.-	-N.A.-	-N.A.-	262.00 282.5	200.00 .2
6	1/16/69	1045	SS	4.00	1.4	38.00 47.8	471.00 592.5	104.00 130.8	22	-N.A.-	-N.A.-	-N.A.-	-N.A.-	-N.A.-	190.00 239.0	-N.A.-
7	1/16/69	14 0	SS	4.00	5.0	-N.A.-	-N.A.-	-N.A.-		-N.A.-	-N.A.-	-N.A.-	-N.A.-	-N.A.-	34.00 152.8	-N.A.-
8	1/16/69	17 0	SS	1.00	.9	-N.A.-	-N.A.-	-N.A.-		-N.A.-	-N.A.-	-N.A.-	-N.A.-	-N.A.-	34.00 6.9	-N.A.-
9	1/16/69	18 0	SS	1.00	.4	33.00 3.0	204.00 18.3	68.00 6.1	33	-N.A.-	-N.A.-	-N.A.-	-N.A.-	-N.A.-	30.00 2.7	-N.A.-
10	1/16/69	19 0	SS	1.00	.2	-N.A.-	-N.A.-	-N.A.-		-N.A.-	-N.A.-	-N.A.-	-N.A.-	-N.A.-	18.00 .8	130.00 .0
11	2/ 4/69	16 0	SS	1.00	2.3	-N.A.-	-N.A.-	-N.A.-		-N.A.-	-N.A.-	-N.A.-	-N.A.-	-N.A.-	1731.00 894.4	199.00 .1
12	2/ 4/69	17 0	SS	-1.00	N.A.	-N.A.-	-N.A.-	-N.A.-		-N.A.-	-N.A.-	-N.A.-	-N.A.-	-N.A.-	1734.00	253.00
13	2/ 4/69	18 0	SS	-1.00	N.A.	-N.A.-	-N.A.-	-N.A.-		-N.A.-	-N.A.-	-N.A.-	-N.A.-	-N.A.-	1486.00	163.00
14	2/ 5/69	15 0	SS	1.00	2.8	-N.A.-	358.00 225.2	93.00 58.5	26	-N.A.-	-N.A.-	-N.A.-	-N.A.-	-N.A.-	1703.00 1071.2	-N.A.-
15	2/ 5/69	16 0	SS	1.00	2.0	-N.A.-	-N.A.-	-N.A.-		-N.A.-	-N.A.-	-N.A.-	-N.A.-	-N.A.-	691.00 310.5	-N.A.-
16	2/ 5/69	17 0	SS	1.00	1.2	-N.A.-	-N.A.-	-N.A.-		-N.A.-	-N.A.-	-N.A.-	-N.A.-	-N.A.-	678.00 182.8	-N.A.-
17	2/ 5/69	18 0	SS	1.00	.7	-N.A.-	-N.A.-	-N.A.-		-N.A.-	-N.A.-	-N.A.-	-N.A.-	-N.A.-	716.00 112.6	-N.A.-

TABLE 25 CON'T

DES MOINES,IOWA,STORM WATER POLLUTION CONTROL STUDY
STATION S-3 CUMMINS PARKWAY STORM SEWER

FWPCA CONTRACT NO 14-12-402

ITEM	DATE	TIME	TYPE	DUR HOUR	FLOW CFS	BOD MG/L LBS	SUSPENDED SOLIDS TOTAL MG/L LBS	SUSPENDED SOLIDS VOLATILE MG/L LBS	SUSPENDED SOLIDS VOLATILE PER CENT	NITROGEN AMMONIA MG/L LBS	NITROGEN NITRITE MG/L LBS	NITROGEN NITRATE MG/L LBS	PHOSPHATES TOTAL MG/L LBS	PHOSPHATES SOLUABLE MG/L LBS	CHLORIDES MG/L LBS	CHROMIUM UG/L LBS
18	2/21/69	1625	SS	1.00	.2	-N.A.-	931.00 41.8	214.00 9.6	23	-N.A.-	-N.A.-	-N.A.-	-N.A.-	-N.A.-	2724.00 122.4	76.00 .0
19	2/22/69	1130	SS	1.00	.1	-N.A.-	120.00 2.9	30.00 .7	23	-N.A.-	-N.A.-	-N.A.-	-N.A.-	-N.A.-	549.00 13.2	164.00 .0
20	2/24/69	1510	SS	1.00	2.0	-N.A.-	1025.00 460.5	195.00 87.6	19	3.32 1.5	.00 .0	.19 .1	-N.A.-	-N.A.-	-N.A.-	-N.A.-
21	2/25/69	11 0	CS	9.00	5.0	153.00 1546.6	1212.00 12251.9	182.00 1839.8	15	.86 8.7	.01 .1	1.02 10.3	.26 2.6	.25 2.5	56.00 566.1	345.00 3.5
22	2/26/69	1330	CS	7.00	3.7	125.00 727.3	357.00 2077.1	-N.A.-		2.66 15.5	.01 .0	.65 3.8	-N.A.-	-N.A.-	34.00 197.8	-N.A.-
23	3/ 1/69	15 0	CS	2.00	2.2	47.00 46.5	-N.A.-	-N.A.-		.69 .7	.01 .0	4.49 4.4	-N.A.-	.05 .0	251.00 248.1	167.00 .2
24	3/ 2/69	13 0	SS	1.00	.3	-N.A.-	-N.A.-	-N.A.-		-N.A.-	-N.A.-	-N.A.-	-N.A.-	-N.A.-	201.00 13.5	47.00 .0
25	3/ 2/69	14 0	SS	1.00	.9	-N.A.-	-N.A.-	-N.A.-		-N.A.-	-N.A.-	-N.A.-	-N.A.-	-N.A.-	179.00 36.2	-N.A.-
26	3/ 2/69	15 0	SS	1.00	1.1	-N.A.-	-N.A.-	-N.A.-		-N.A.-	-N.A.-	-N.A.-	-N.A.-	-N.A.-	126.00 31.1	139.00 .0
27	3/ 2/69	16 0	SS	1.00	.9	-N.A.-	-N.A.-	-N.A.-		-N.A.-	-N.A.-	-N.A.-	-N.A.-	-N.A.-	159.00 32.1	-N.A.-
28	6/22/69	1540	SR	.25	.3	154.00 2.6	9.00 .2	6.00 .1	67	-N.A.-	-N.A.-	-N.A.-	-N.A.-	-N.A.-	-N.A.-	-N.A.-
29	6/22/69	16 0	SR	.75	3.8	166.00 186.3	1146.00 733.7	175.00 112.0	15	-N.A.-	-N.A.-	-N.A.-	-N.A.-	-N.A.-	-N.A.-	-N.A.-
30	6/22/69	17 0	SR	1.00	5.8	105.00 136.8	476.00 620.2	60.00 78.2	13	-N.A.-	-N.A.-	-N.A.-	-N.A.-	-N.A.-	-N.A.-	-N.A.-
31	6/22/69	18 0	SR	1.00	4.2	111.00 104.7	340.00 320.8	102.00 96.2	30	-N.A.-	-N.A.-	-N.A.-	-N.A.-	-N.A.-	-N.A.-	-N.A.-
32	6/22/69	19 0	SR	1.00	9.4	111.00 234.4	307.00 648.3	43.00 90.8	14	-N.A.-	-N.A.-	-N.A.-	-N.A.-	-N.A.-	-N.A.-	-N.A.-
33	6/26/69	12 0	SR	.25	7.5	14.00 5.9	14.00 5.9	6.00 2.5	43	-N.A.-	-N.A.-	-N.A.-	-N.A.-	-N.A.-	-N.A.-	-N.A.-
34	6/26/69	1215	SR	.25	4.8	-N.A.-	-N.A.-	-N.A.-		.70 .2	.01 .0	.20 .1	.79 .2	.30 .1	-N.A.-	-N.A.-

TABLE 25 CON'T

DES MOINES,IOWA,STORM WATER POLLUTION CONTROL STUDY
STATION S-3 CUMMINS PARKWAY STORM SEWER

FWPCA CONTRACT NO 14-12-402

ITEM	DATE	TIME	TYPE	DUR HOUR	FLOW CFS	BOD MG/L LBS	SUSPENDED SOLIDS TOTAL MG/L LBS	SUSPENDED SOLIDS VOLATILE MG/L LBS	SUSPENDED SOLIDS VOLATILE PER CENT	NITROGEN AMMONIA MG/L LBS	NITROGEN NITRITE MG/L LBS	NITRATE MG/L LBS	PHOSPHATES TOTAL MG/L LBS	PHOSPHATES SOLUABLE MG/L LBS	CHLORIDES MG/L LBS	CHROMIUM UG/L LBS
35	6/26/69	17 0	SR	.25	5.8	-N.A.-	3170.00 1032.6	276.00 89.9	9	-N.A.-	-N.A.-	-N.A.-	-N.A.-	-N.A.-	-N.A.-	-N.A.-
36	6/26/69	1715	SP	.25	47.9	23.00 61.9	1260.00 3389.5	160.00 430.4	13	-N.A.-	-N.A.-	-N.A.-	-N.A.-	-N.A.-	-N.A.-	-N.A.-
37	6/26/69	1730	SR	.25	17.1	22.00 21.1	604.00 580.8	82.00 78.7	14	.70 .7	.02 .0	.29 .3	1.76 1.7	.66 .6	-N.A.-	-N.A.-
38	8/ 7/69	730	SR	.50	1.1	17.00 2.1	-N.A.-	-N.A.-		.03 .1	.08 .0	-N.A.-	-N.A.-	-N.A.-	-N.A.-	-N.A.-
39	8/ 7/69	8 0	SR	.50	.2	56.00 1.3	-N.A.-	-N.A.-		-N.A.-	-N.A.-	-N.A.-	-N.A.-	-N.A.-	-N.A.-	-N.A.-
40	8/20/69	10 0	CR	4.00	2.7	29.00 70.4	99.00 143.1	12.00 29.1	20	1.94 4.7	.02 .0	.54 1.3	.64 1.6	.59 1.4	-N.A.-	-N.A.-
41	9/ 4/69	830	SR	.25	6.9	27.00 13.5	521.00 260.4	90.00 45.0	17	-N.A.-	-N.A.-	-N.A.-	-N.A.-	-N.A.-	-N.A.-	-N.A.-
42	9/ 4/69	845	SR	.25	0.0	-N.A.-	-N.A.-	-N.A.-		2.62 0.0	.06 0.0	.90 0.0	1.42 0.0	.89 0.0	-N.A.-	-N.A.-
43	9/ 4/69	9 0	SR	.25	0.0	53.00 0.0	-N.A.-	-N.A.-		-N.A.-	-N.A.-	-N.A.-	-N.A.-	-N.A.-	-N.A.-	-N.A.-
44	9/13/69	2045	CR	4.00	1.5	27.00 36.4	50.00 67.4	17.00 22.9	34	.78 1.1	.06 .1	1.02 1.4	.45 .6	.38 .5	-N.A.-	-N.A.-
45	9/23/69	21 0	SR	.25	10.2	31.00 17.0	29.00 16.6	-N.A.-		-N.A.-	-N.A.-	-N.A.-	-N.A.-	.40 .2	-N.A.-	-N.A.-
46	10/ 2/69	830	SR	.25	0.0	56.00 0.0	-N.A.-	-N.A.-		-N.A.-	-N.A.-	-N.A.-	-N.A.-	-N.A.-	-N.A.-	-N.A.-
47	10/ 2/69	845	SR	.25	0.0	-N.A.-	-N.A.-	-N.A.-		2.47 0.0	.01 0.0	4.61 0.0	1.16 0.0	.89 0.0	-N.A.-	-N.A.-

TABLE 25 CON'T

DES MOINES, IOWA, STORM WATER POLLUTION CONTROL STUDY
STATION 0-11 20TH ST. STORM SEWER AT DEAN LAKE

FWPCA CONTRACT NO 14-12-402

ITEM	DATE	TIME	TYPE	DUR HOUR	FLOW CFS	BOD MG/L LBS	SUSPENDED SOLIDS TOTAL MG/L LBS	SUSPENDED SOLIDS VOLATILE MG/L LBS	SUSPENDED SOLIDS VOLATILE PER CENT	NITROGEN AMMONIA MG/L LBS	NITROGEN NITRITE MG/L LBS	NITRATE MG/L LBS	PHOSPHATES TOTAL MG/L LBS	PHOSPHATES SOLUABLE MG/L LBS	CHLORIDES MG/L LBS	CHROMIUM UG/L LBS
1	2/22/69	2330	SS	1.00	.9	-N.A.-	162.00 32.8	57.00 11.5	35	-N.A.-	-N.A.-	-N.A.-	-N.A.-	-N.A.-	292.00 59.0	139.00 .0
2	2/24/69	1130	SS	1.00	.6	-N.A.-	41.00 5.5	18.00 2.4	44	1.88 .3	.20 .0	1.89 .3	-N.A.-	-N.A.-	139.00 18.7	-N.A.-
3	2/24/69	12 0	CS	24.00	2.6	69.00 467.2	520.00 7401.3	185.00 2593.2	35	3.60 50.5	.03 .4	.61 8.6	-N.A.-	.27 3.8	-N.A.-	-N.A.-
4	2/25/69	12 0	SS	1.00	3.0	127.00 85.6	597.00 402.3	197.00 132.8	33	-N.A.-	-N.A.-	-N.A.-	-N.A.-	-N.A.-	114.00 76.8	-N.A.-
5	2/25/69	13 0	SS	1.00	15.0	-N.A.-	-N.A.-	-N.A.-		.40 1.3	.00 .0	.31 1.0	-N.A.-	-N.A.-	86.00 289.8	-N.A.-
6	2/25/69	14 0	CS	4.00	29.4	-N.A.-	-N.A.-	-N.A.-		1.80 47.6	.01 .3	.27 7.1	-N.A.-	-N.A.-	42.00 1109.5	-N.A.-
7	2/25/69	19 0	SS	1.00	10.0	143.00 321.2	-N.A.-	-N.A.-		.66 1.5	.03 .1	1.06 2.4	-N.A.-	-N.A.-	-N.A.-	-N.A.-
8	2/25/69	20 0	SS	1.00	5.9	-N.A.-	167.00 221.3	63.00 83.5	38	-N.A.-	-N.A.-	-N.A.-	-N.A.-	-N.A.-	37.00 49.0	-N.A.-
9	2/26/69	21 0	CS	15.00	1.7	129.00 739.0	49.00 280.7	35.00 200.5	71	1.14 6.5	.01 .1	1.21 6.9	.26 1.5	.23 1.3	42.00 240.6	137.00 .8
10	2/27/69	12 0	CS	22.00	5.1	51.00 1285.4	207.00 5217.4	79.00 1991.2	38	4.57 115.2	.01 .2	1.04 26.2	.29 7.3	.28 7.1	35.00 882.2	21.00 .5
11	2/28/69	1445	SS	1.00	2.2	-N.A.-	-N.A.-	-N.A.-		-N.A.-	-N.A.-	-N.A.-	-N.A.-	-N.A.-	514.00 254.0	-N.A.-
12	3/ 1/69	14 0	CS	11.00	4.7	53.00 615.5	829.00 9627.9	216.00 2508.6	26	.78 9.1	.01 .1	1.02 11.8	.18 2.1	.03 .3	-N.A.-	66.00 .8
13	3/ 1/69	13 0	SS	1.00	3.8	-N.A.-	-N.A.-	-N.A.-		-N.A.-	-N.A.-	-N.A.-	-N.A.-	-N.A.-	306.00 261.2	76.00 .1
14	3/ 2/69	11 0	CS	12.00	1.7	35.00 160.4	287.00 1315.2	-N.A.-		1.94 8.9	.01 .0	.86 3.9	.09 .4	.07 .3	-N.A.-	47.00 .2
15	3/ 2/69	11 0	SS	1.00	.3	-N.A.-	-N.A.-	-N.A.-		-N.A.-	-N.A.-	-N.A.-	-N.A.-	-N.A.-	131.00 8.0	-N.A.-
16	3/ 2/69	14 0	SS	1.00	2.7	-N.A.-	-N.A.-	-N.A.-		-N.A.-	-N.A.-	-N.A.-	-N.A.-	-N.A.-	111.00 67.3	-N.A.-
17	4/14/69	1015	CR	12.00	4.6	37.00 458.4	411.00 5096.5	126.00 1562.4	31	.43 5.3	.00 .0	1.18 14.6	2.32 28.8	.46 5.7	-N.A.-	-N.A.-

TABLE 25 CON'T

DES MOINES, IOWA, STORM WATER POLLUTION CONTROL STUDY
STATION 0-11 20TH ST. STORM SEWER AT GRAYS LAKE

FWPCA CONTRACT NO 14-12-402

ITEM	DATE	TIME	TYPE	DUR HOUR	FLOW CFS	BOD MG/L LBS	SUSPENDED SOLIDS TOTAL MG/L LBS	SUSPENDED SOLIDS VOLATILE MG/L LBS	SUSPENDED SOLIDS VOLATILE PER CENT	NITROGEN AMMONIA MG/L LBS	NITROGEN NITRITE MG/L LBS	NITROGEN NITRATE MG/L LBS	PHOSPHATES TOTAL MG/L LBS	PHOSPHATES SOLUABLE MG/L LBS	CHLORIDES MG/L LBS	CHROMIUM UG/L LBS
18	4/17/69	1845	CR	23.00	4.5	12.00 341.0	234.00 6649.6	71.00 2017.6	30	.06 1.7	.15 4.3	1.22 34.7	1.37 38.9	1.35 38.4	-N.A.-	-N.A.-
19	5/ 7/69	1730	CR	8.00	30.0	41.00 1671.3	381.00 20541.1	83.00 4474.8	22	.32 17.3	.01 .3	.67 36.1	2.08 112.1	.60 32.3	-N.A.-	-N.A.-
20	5/ 8/69	130	CR	8.00	6.6	22.00 260.4	47.00 1150.5	26.00 332.1	29	.66 7.8	.01 .1	1.80 21.3	1.50 17.8	1.14 13.5	-N.A.-	-N.A.-
21	5/ 7/69	1730	SR	1.00	6.2	137.00 190.8	-N.A.-	-N.A.-		-N.A.-	-N.A.-	-N.A.-	-N.A.-	-N.A.-	-N.A.-	-N.A.-
22	5/ 7/69	1830	SR	1.00	20.9	38.00 178.4	-N.A.-	-N.A.-		-N.A.-	-N.A.-	-N.A.-	-N.A.-	-N.A.-	-N.A.-	-N.A.-
23	5/ 7/69	1930	SR	1.00	33.9	-N.A.-	540.00 6396.8	224.00 1705.8	27	-0.00 0.0	-0.00 0.0	-0.00 0.0	-0.00 0.0	-0.00 0.0	-0.00 0.0	-0.00 0.0
24	5/ 7/69	2330	SR	1.00	29.4	28.00 184.9	-N.A.-	-N.A.-		-N.A.-	-N.A.-	-N.A.-	-N.A.-	-N.A.-	-N.A.-	-N.A.-
25	5/ 8/69	030	SR	1.00	17.2	-N.A.-	267.00 1031.6	76.00 293.6	28	-N.A.-	-N.A.-	-N.A.-	-N.A.-	-N.A.-	-N.A.-	-N.A.-
26	5/17/69	3 0	CR	10.00	14.1	51.00 1615.4	880.00 27873.3	174.00 5511.3	20	.49 15.5	.03 1.0	.22 7.0	4.21 133.3	2.36 74.8	-N.A.-	-N.A.-
27	5/26/69	8 0	CR	5.00	12.0	77.00 1037.8	880.00 11861.0	204.00 2749.6	23	1.40 18.9	.01 .1	1.30 17.5	2.61 35.2	.21 2.8	-N.A.-	-N.A.-
28	7/ 8/69	10 0	CR	16.00	1.3	14.00 65.4	40.00 186.9	15.00 70.1	37	27.78 129.8	.04 .2	1.96 9.2	.82 3.8	.20 .9	-N.A.-	-N.A.-
29	7/ 9/69	2 0	CR	3.00	6.8	25.00 114.6	358.00 1640.6	72.00 330.0	20	.58 2.7	.06 .3	1.67 7.7	2.63 12.1	.52 2.4	-N.A.-	-N.A.-
30	7/ 9/69	5 0	CR	5.00	39.0	28.00 1226.5	658.00 28823.6	121.00 5300.4	18	4.74 207.6	.01 .4	.68 29.8	2.64 115.6	.43 18.8	-N.A.-	-N.A.-
31	7/17/69	930	SR	.50	45.8	12.00 61.7	202.00 1039.1	57.00 293.2	28	.56 2.9	.01 .1	.41 2.1	-N.A.-	.50 2.6	-N.A.-	-N.A.-
32	7/17/69	10 0	CR	3.00	16.5	20.00 222.4	135.00 1501.2	18.00 200.2	13	.49 5.4	.01 .1	.52 5.8	.93 10.3	.73 8.1	-N.A.-	-N.A.-
33	7/23/69	725	SR	.50	67.3	38.00 287.2	588.00 4444.8	146.00 1103.6	25	-N.A.-	-N.A.-	-N.A.-	-N.A.-	-N.A.-	-N.A.-	-N.A.-
34	7/23/69	735	SR	.50	67.3	-N.A.-	-N.A.-	-N.A.-		.68 5.1	.08 .6	1.46 11.0	2.66 20.1	.48 3.6	-N.A.-	-N.A.-

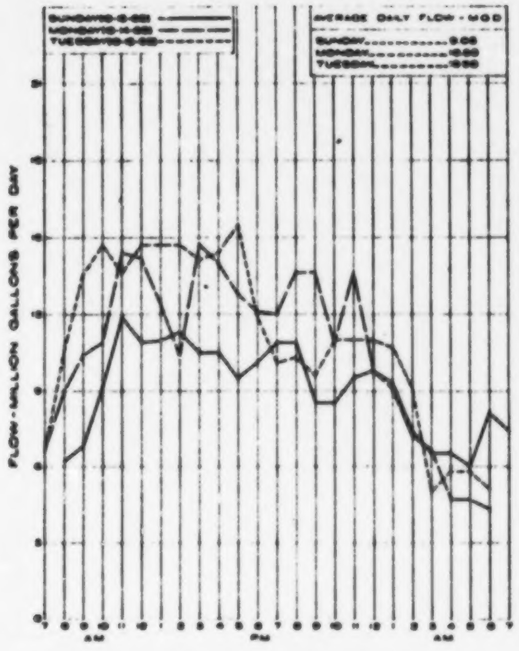
TABLE 25 CON'T

DES MOINES,IOWA,STORM WATER POLLUTION CONTROL STUDY
STATION U-11 20TH ST. STORM SEWER AT DEAN LAKE

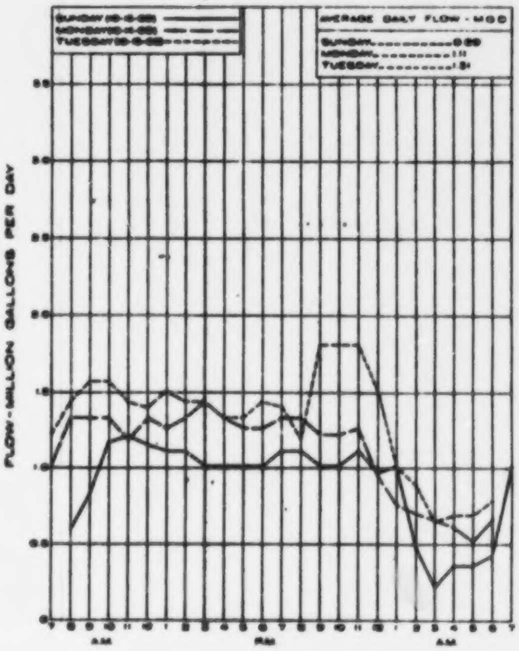
FWPCA CONTRACT NO 14-12-402

							SUSPENDED SOLIDS			NITROGEN			PHOSPHATES		CHLORIDES	CHROMIUM
ITEM	DATE	TIME	TYPE	DUR HOUR	FLOW CFS	BOD MG/L LBS	TOTAL MG/L LBS	VOLATILE MG/L LBS	PER CENT	AMMONIA MG/L LBS	NITRITE MG/L LBS	NITRATE MG/L LBS	TOTAL MG/L LBS	SOLUABLE MG/L LBS	MG/L LBS	UG/L LBS
35	7/23/69	815	SR	.50	40.8	35.00 160.4	808.00 3702.8	122.00 559.1	15	-N.A.-	-N.A.-	-N.A.-	-N.A.-	-N.A.-	-N.A.-	-N.A.-
36	7/23/69	825	SR	.50	27.2	-N.A.-	-N.A.-	-N.A.-		.55 1.7	.05 .2	.80 2.4	1.86 5.7	.52 1.6	-N.A.-	-N.A.-
37	7/23/69	850	SR	1.00	20.9	23.00 108.0	356.00 1671.4	66.00 309.9	19	-N.A.-	-N.A.-	-N.A.-	-N.A.-	-N.A.-	-N.A.-	-N.A.-
38	9/ 4/69	830	SR	.50	3.2	39.00 14.0	196.00 70.4	54.00 19.4	28	-N.A.-	-N.A.-	-N.A.-	-N.A.-	-N.A.-	-N.A.-	-N.A.-
39	9/ 4/69	9 0	SR	.50	6.2	-N.A.-	-N.A.-	-N.A.-		2.42 1.7	.08 .1	1.22 .8	1.47 1.0	.48 .3	-N.A.-	-N.A.-
40	9/ 4/69	930	SR	.50	8.0	65.00 58.4	280.00 251.6	64.00 57.5	23	-N.A.-	-N.A.-	-N.A.-	-N.A.-	-N.A.-	-N.A.-	-N.A.-
41	9/ 4/69	10 0	SR	.50	7.6	-N.A.-	-N.A.-	-N.A.-		1.12 1.0	.04 .0	1.22 1.0	-N.A.-	1.01 .9	-N.A.-	-N.A.-
42	9/ 4/69	1045	SR	.50	6.9	34.00 26.4	-N.A.-	-N.A.-		-N.A.-	-N.A.-	-N.A.-	-N.A.-	-N.A.-	-N.A.-	-N.A.-
43	9/ 4/69	1115	SR	.50	4.6	-N.A.-	-N.A.-	-N.A.-		2.33 1.2	.09 .0	.57 .3	3.73 1.9	.55 .3	-N.A.-	-N.A.-
44	9/ 4/69	1145	SR	.50	3.8	47.00 20.1	247.00 105.4	64.00 27.3	26	-N.A.-	-N.A.-	-N.A.-	-N.A.-	-N.A.-	-N.A.-	-N.A.-
45	9/13/69	2030	CR	7.00	2.0	50.00 157.2	95.00 298.8	41.00 128.9	43	.56 1.8	.02 .1	.40 1.3	-N.A.-	.51 1.6	-N.A.-	-N.A.-
46	9/22/69	1515	SR	.50	2.8	88.00 27.7	569.00 178.9	167.00 52.5	29	-N.A.-	-N.A.-	-N.A.-	-N.A.-	-N.A.-	-N.A.-	-N.A.-
47	9/22/69	1530	CR	2.00	18.8	44.00 371.6	487.00 4113.4	119.00 1005.1	24	1.31 11.1	.05 .5	.70 5.9	1.82 15.4	.35 3.0	-N.A.-	-N.A.-
48	9/22/69	1730	CR	3.00	23.6	38.00 604.4	410.00 6520.8	91.00 1447.3	22	1.14 18.1	.03 .5	.79 12.6	1.32 21.0	.33 5.2	-N.A.-	-N.A.-
49	10/30/69	445	CR	9.00	4.1	100.00 828.9	1053.00 8728.5	484.00 4012.0	46	2.57 21.3	.09 .8	.65 5.4	12.00 99.5	.57 4.7	-N.A.-	-N.A.-
50	10/30/69	445	SP	1.00	1.1	100.00 24.7	-N.A.-	-N.A.-		-N.A.-	-N.A.-	-N.A.-	-N.A.-	.60 .1	-N.A.-	-N.A.-
51	10/30/69	1145	SP	1.00	3.8	125.00 106.7	-N.A.-	-N.A.-		-N.A.-	-N.A.-	-N.A.-	-N.A.-	-N.A.-	-N.A.-	-N.A.-

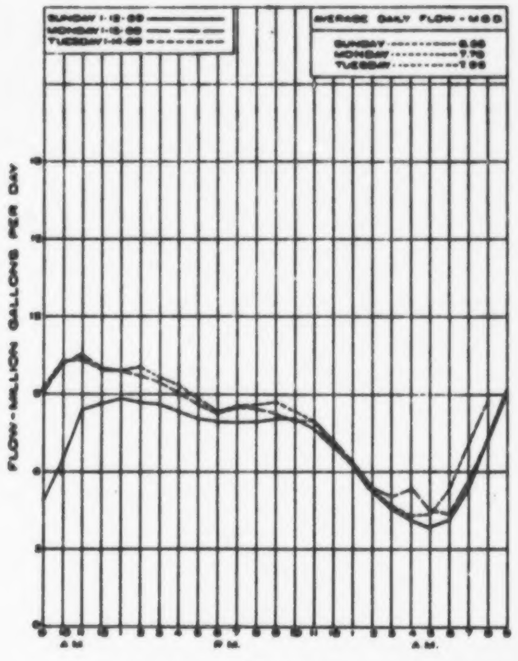
TABLE 25 CON'T



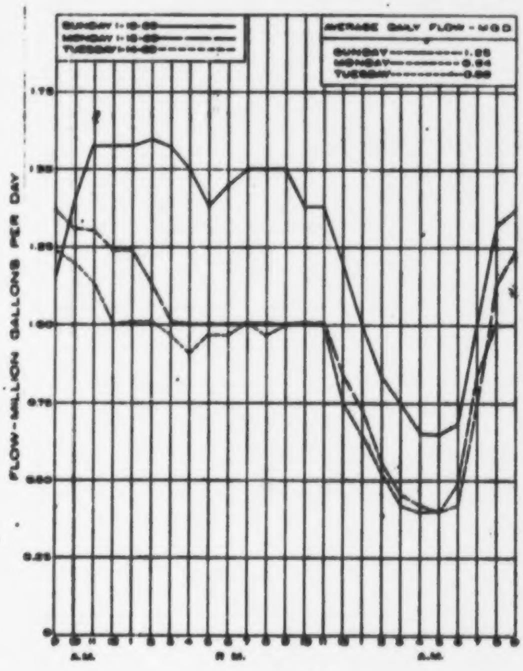
DRY WEATHER SANITARY FLOW
STATION D-1
OCTOBER 13-14-15, 1968



DRY WEATHER SANITARY FLOW
STATION D-1A
OCTOBER 13-14-15, 1968



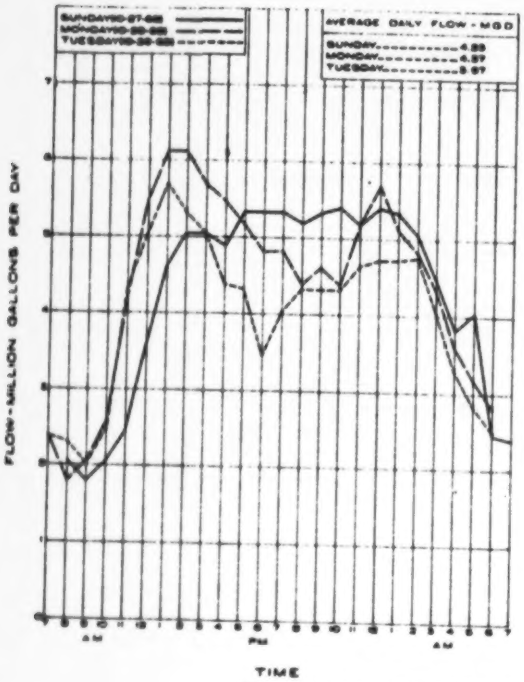
DRY WEATHER SANITARY FLOW
STATION D-1
JANUARY 12-13-14, 1969



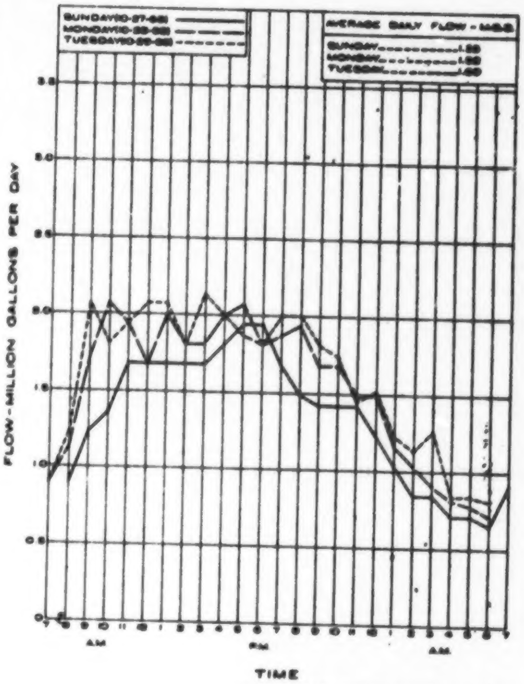
DRY WEATHER SANITARY FLOW
STATION D-1B
JANUARY 12-13-14, 1969

DRY WEATHER SANITARY FLOWS

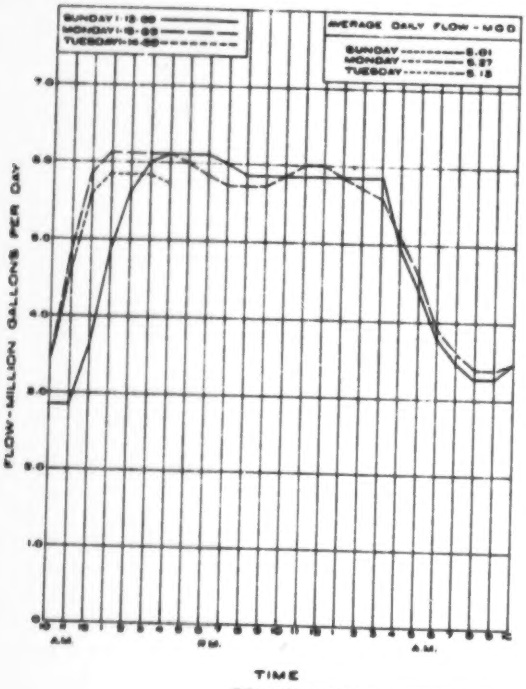
FIGURE 66



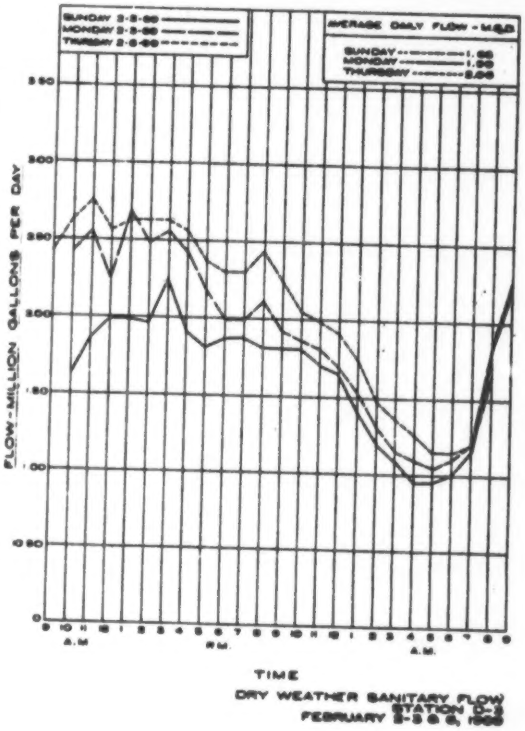
DRY WEATHER SANITARY FLOW
STATION D-2
OCTOBER 27-28-29, 1968



DRY WEATHER SANITARY FLOW
STATION D-3
OCTOBER 27-28-29, 1968

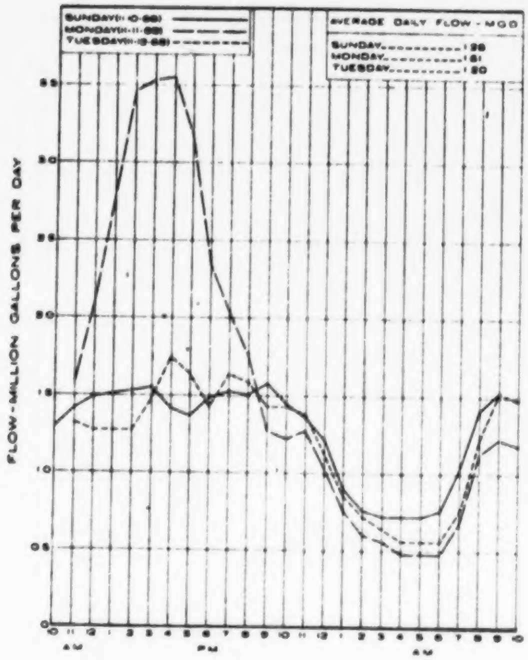


DRY WEATHER SANITARY FLOW
STATION D-2
JANUARY 12-13-14, 1969

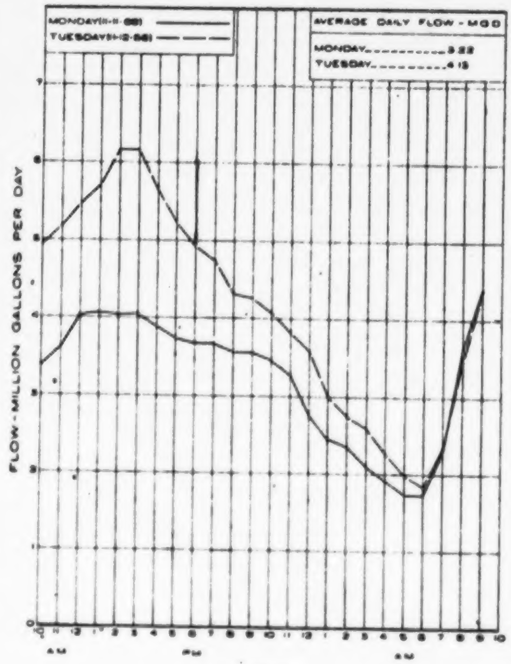


DRY WEATHER SANITARY FLOW
STATION D-3
FEBRUARY 2-3 & 6, 1969

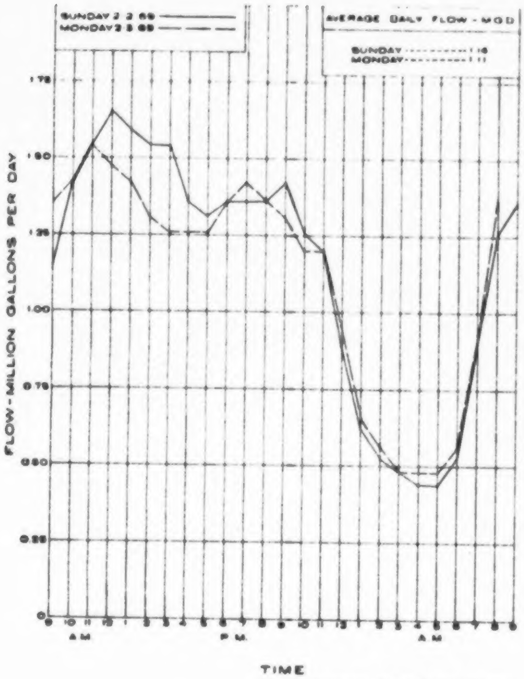
FIGURE 66 CON'T



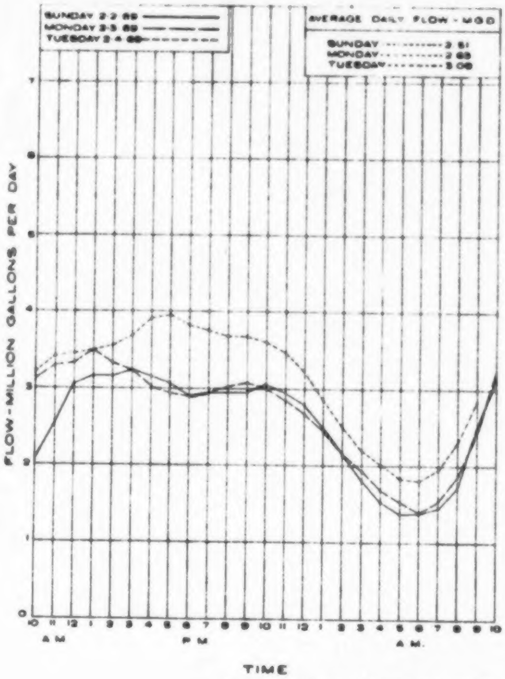
DRY WEATHER SANITARY FLOW
STATION D-5
NOVEMBER 10-11-12, 1968



DRY WEATHER SANITARY FLOW
STATION D-4
NOVEMBER 11 & 12, 1968

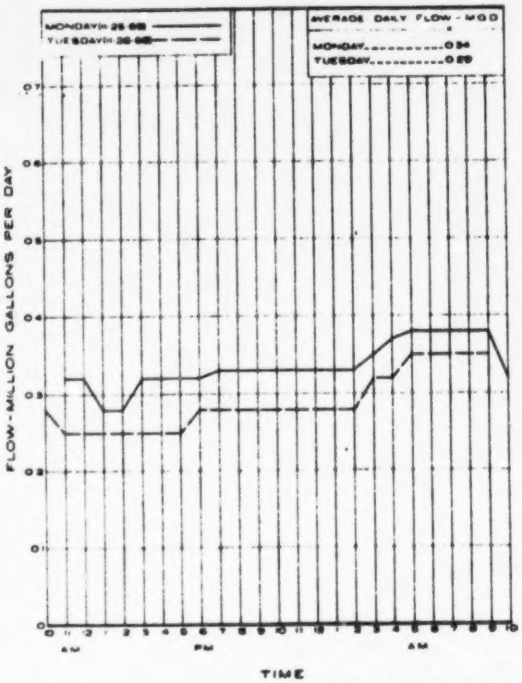


DRY WEATHER SANITARY FLOW
STATION D-5
FEBRUARY 2 & 3, 1969

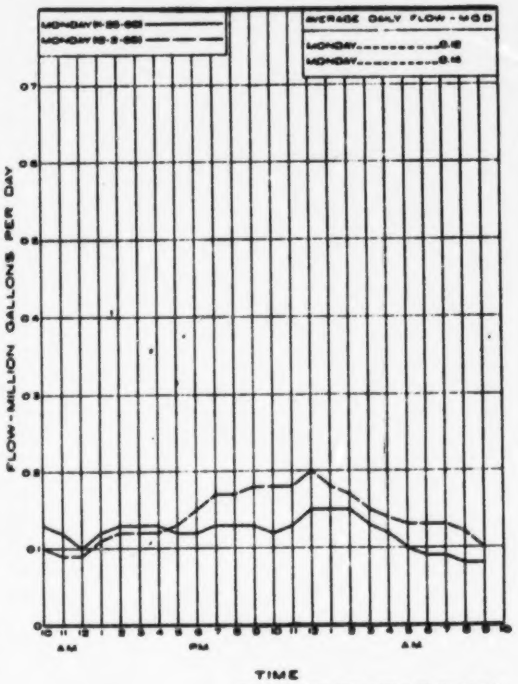


DRY WEATHER SANITARY FLOW
STATION D-9
FEBRUARY 2-3-4, 1969

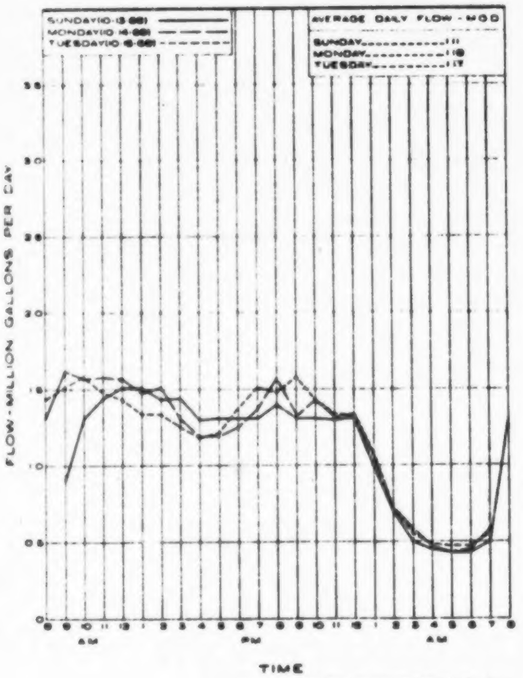
FIGURE 66 CON'T



DRY WEATHER SANITARY FLOW
STATION D-7
NOVEMBER 25 & 26, 1968



DRY WEATHER SANITARY FLOW
STATION D-8
NOVEMBER 25 & DECEMBER 2, 1968



DRY WEATHER SANITARY FLOW
STATION D-19
OCTOBER 13-14-15, 1968

FIGURE 66 CON'T

APPENDIX C

River Sampling Data

DISSOLVED OXYGEN - DIURNAL PATTERNS

STREAM DES MOINES RIVER

DATE FEBRUARY (5-6) 1969 FLOW R-2, 520 C.F.S. R-5, 800 C.F.S.

SAMPLE POINT LEFT QUARTER SAMPLE POINT RIGHT QUARTER

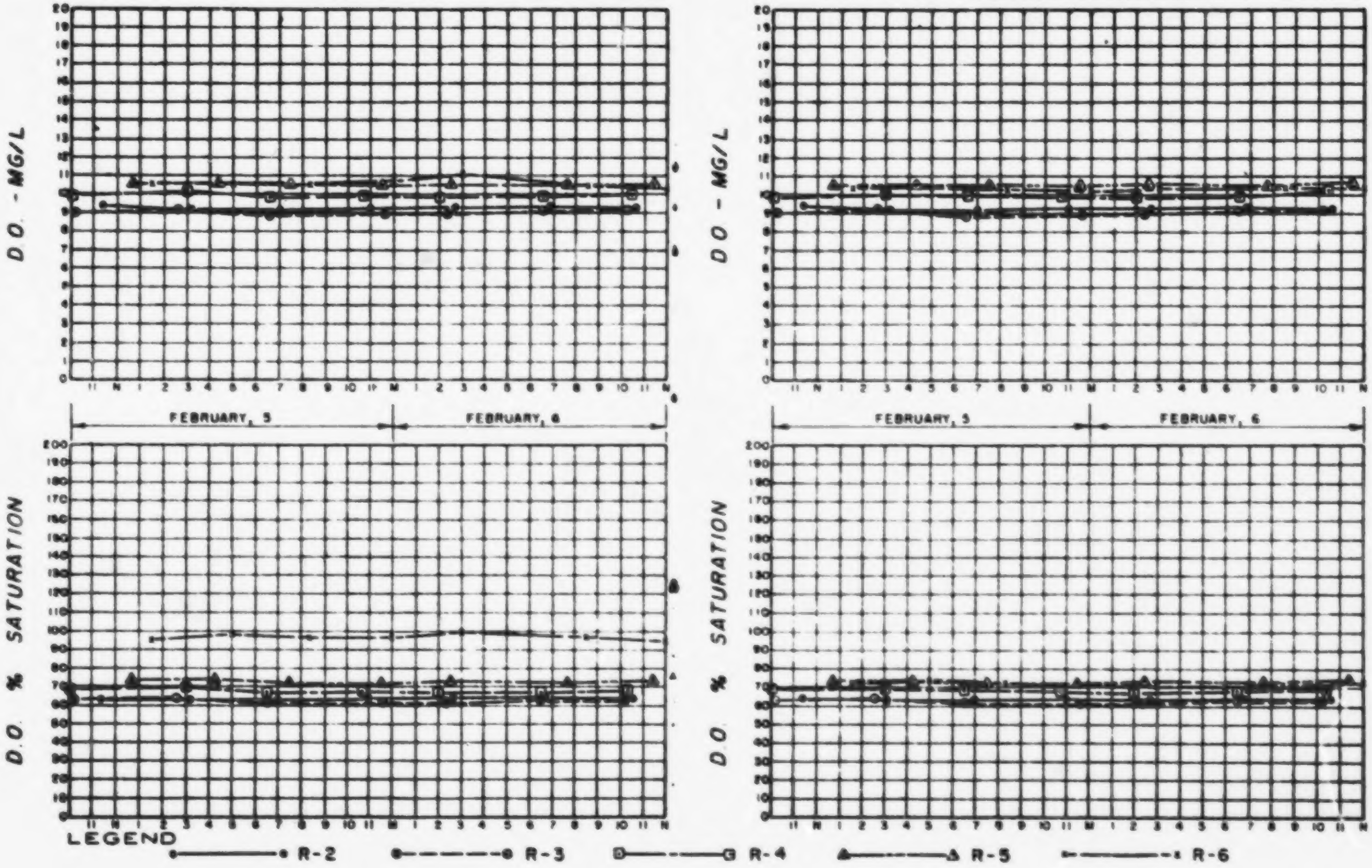


FIGURE 67

DISSOLVED OXYGEN - DIURNAL PATTERNS

STREAM DES MOINES RIVER

DATE JUNE (5-6) 1969

FLOW R-2, 3090 CFS. R-5, 4420 CFS.

SAMPLE POINT LEFT QUARTER

SAMPLE POINT RIGHT QUARTER

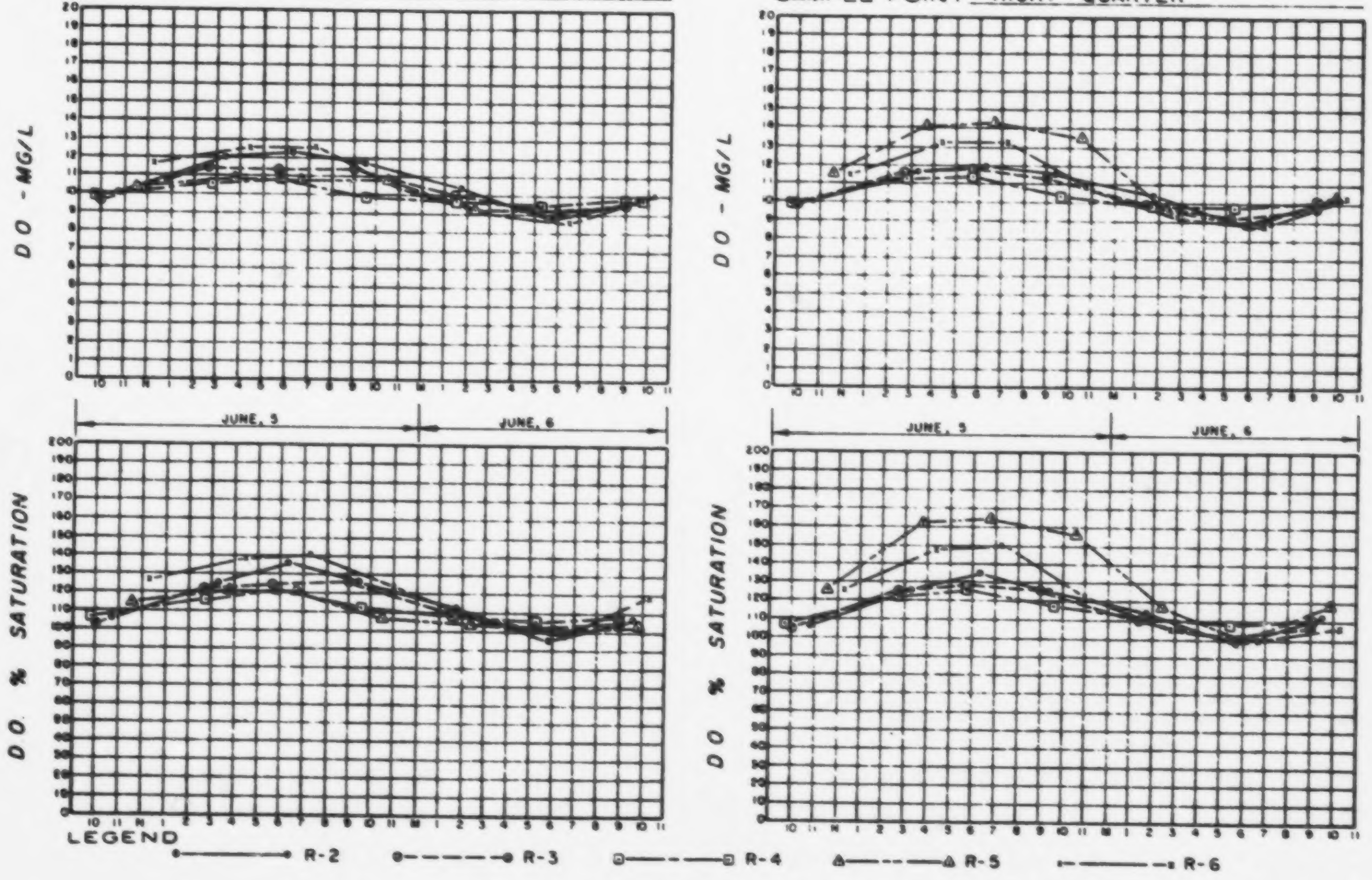


FIGURE 67 CON'T.

DISSOLVED OXYGEN - DIURNAL PATTERNS

STREAM DES MOINES RIVER

DATE AUGUST (20-21) 1969

FLOW R-2, 1580 CFS. R-5, 2820 CFS.

SAMPLE POINT LEFT QUARTER

SAMPLE POINT RIGHT QUARTER

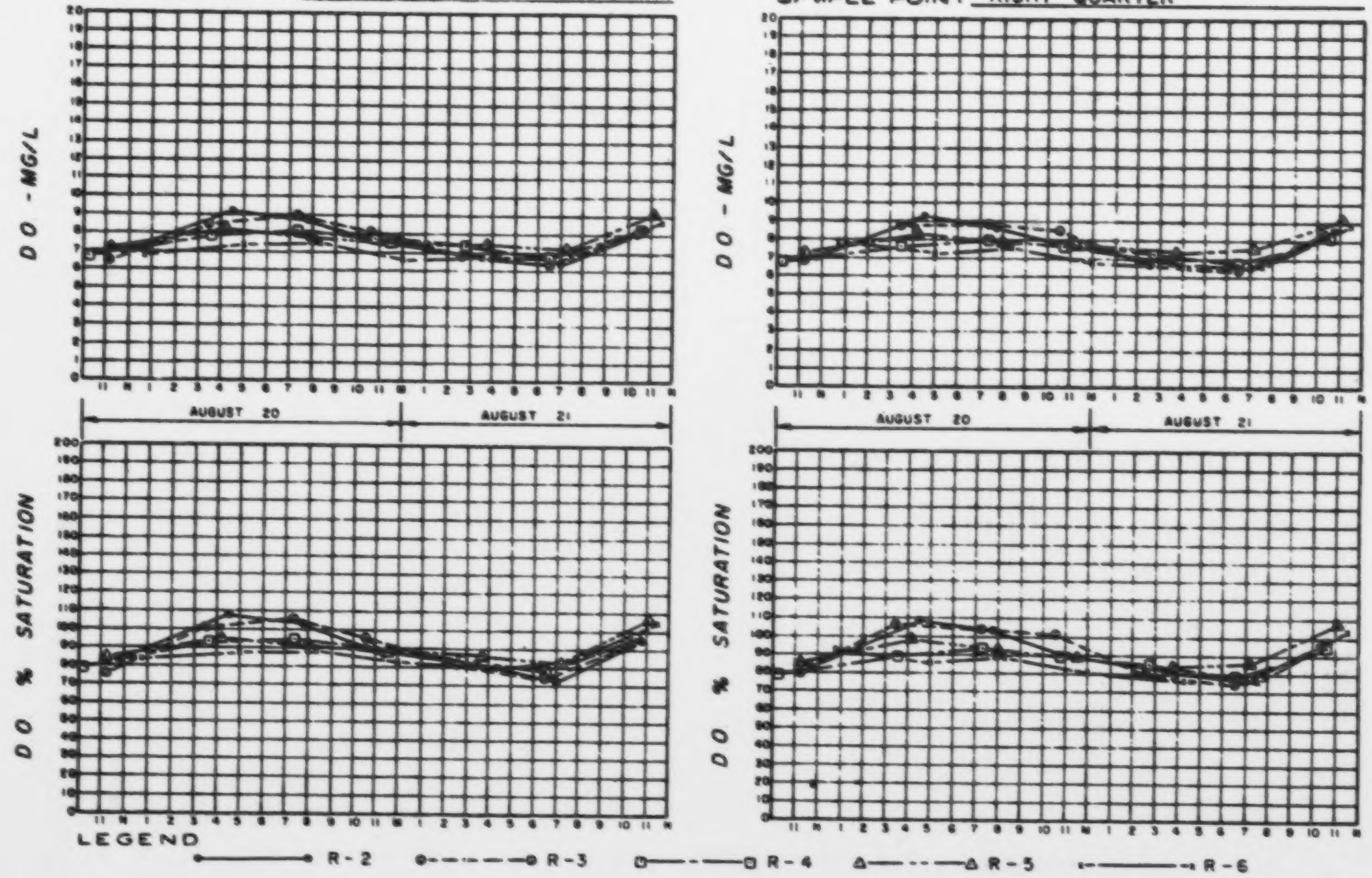


FIGURE 67 CON'T

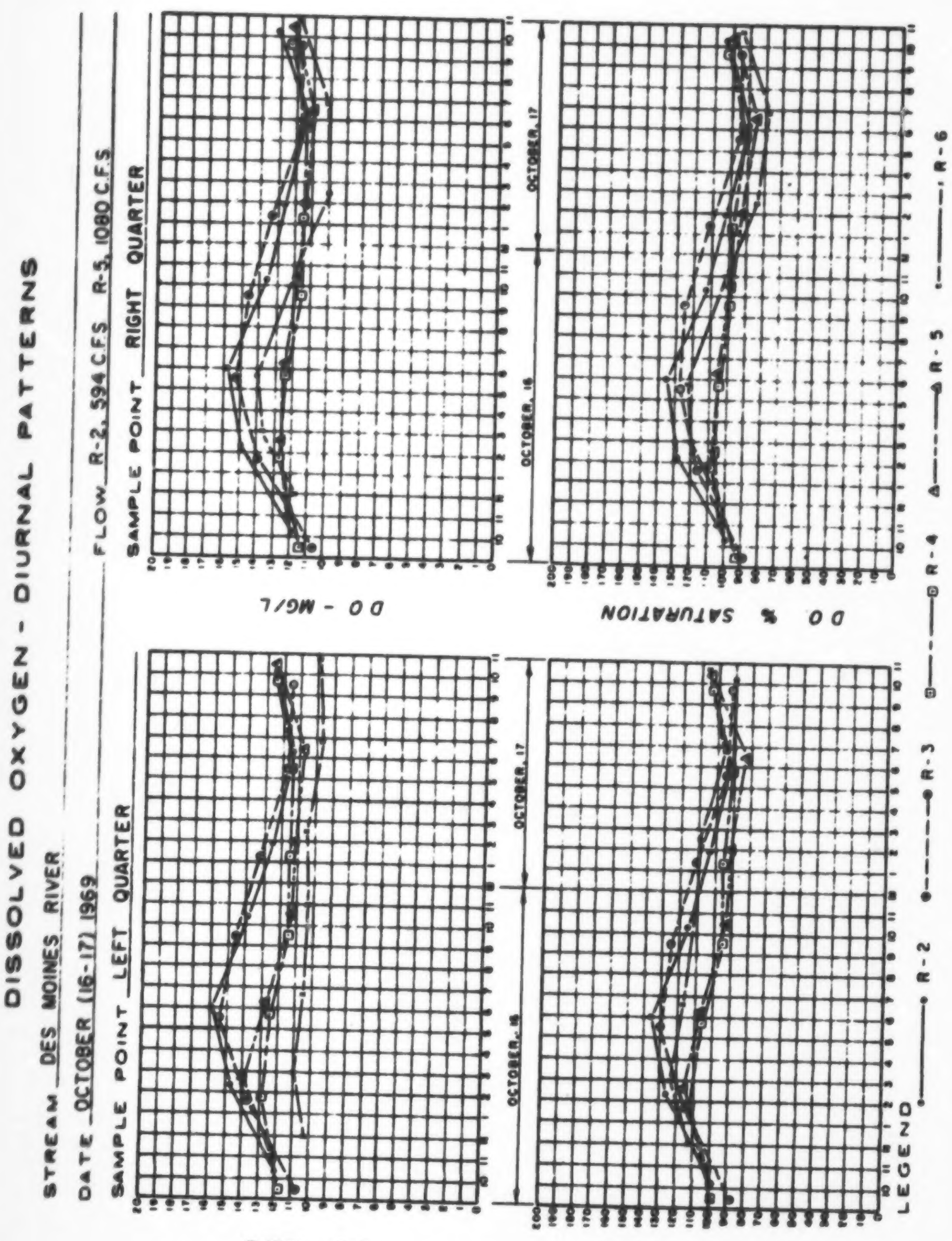


FIGURE 67 CON'T

DISSOLVED OXYGEN - DIURNAL PATTERNS

STREAM RACCOON RIVER

DATE FEBRUARY (5-6) 1969

FLOW 350 CFS

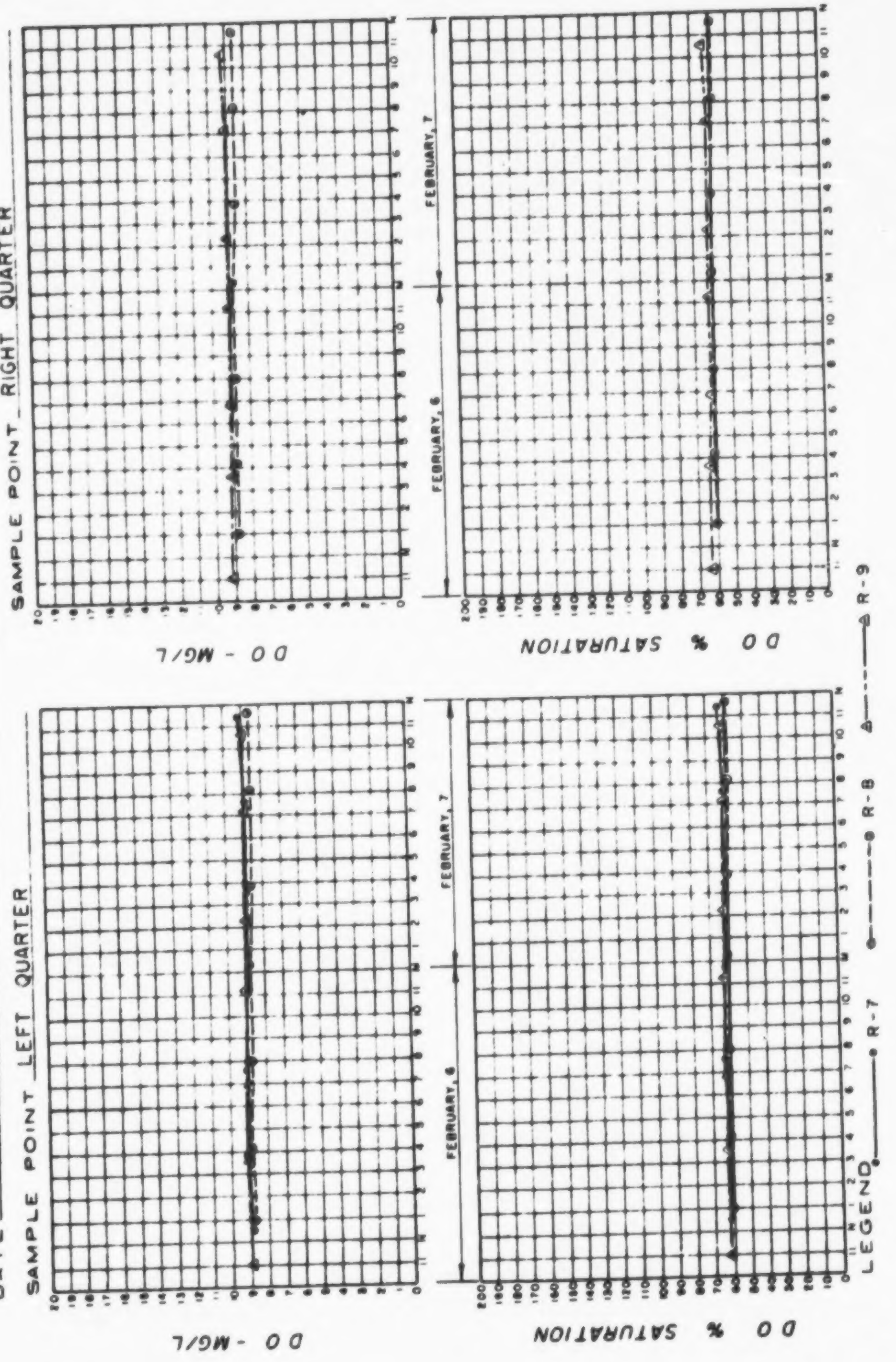


FIGURE 68

DISSOLVED OXYGEN - DIURNAL PATTERNS

STREAM RACCOON RIVER

DATE JUNE (5-6) 1969 FLOW 1200 CFS.

SAMPLE POINT LEFT QUARTER SAMPLE POINT RIGHT QUARTER

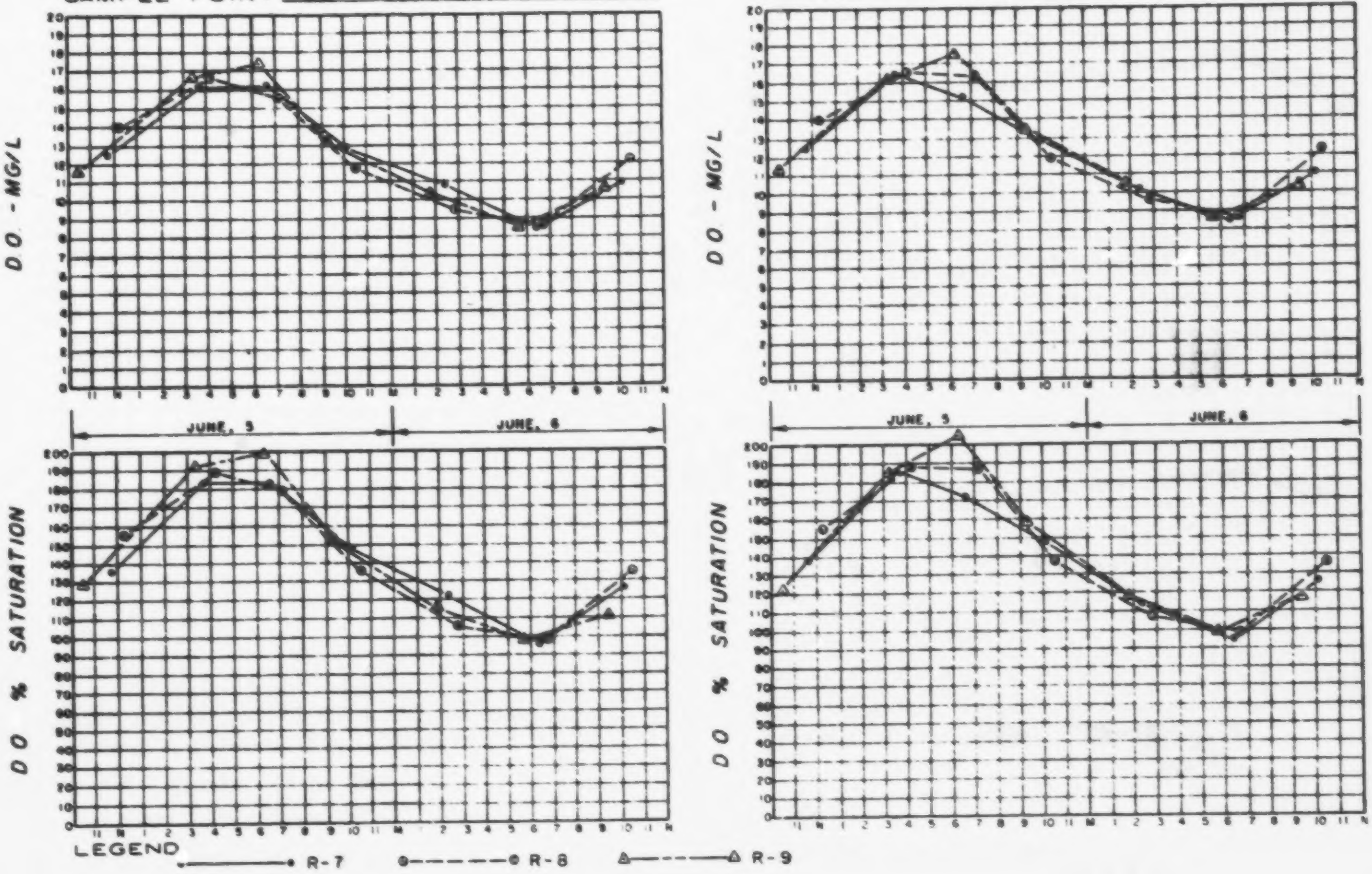


FIGURE 68 CON'T

DISSOLVED OXYGEN - DIURNAL PATTERNS

STREAM RACCOON RIVER

DATE AUGUST (20-21) 1969

FLOW 905 C.F.S.

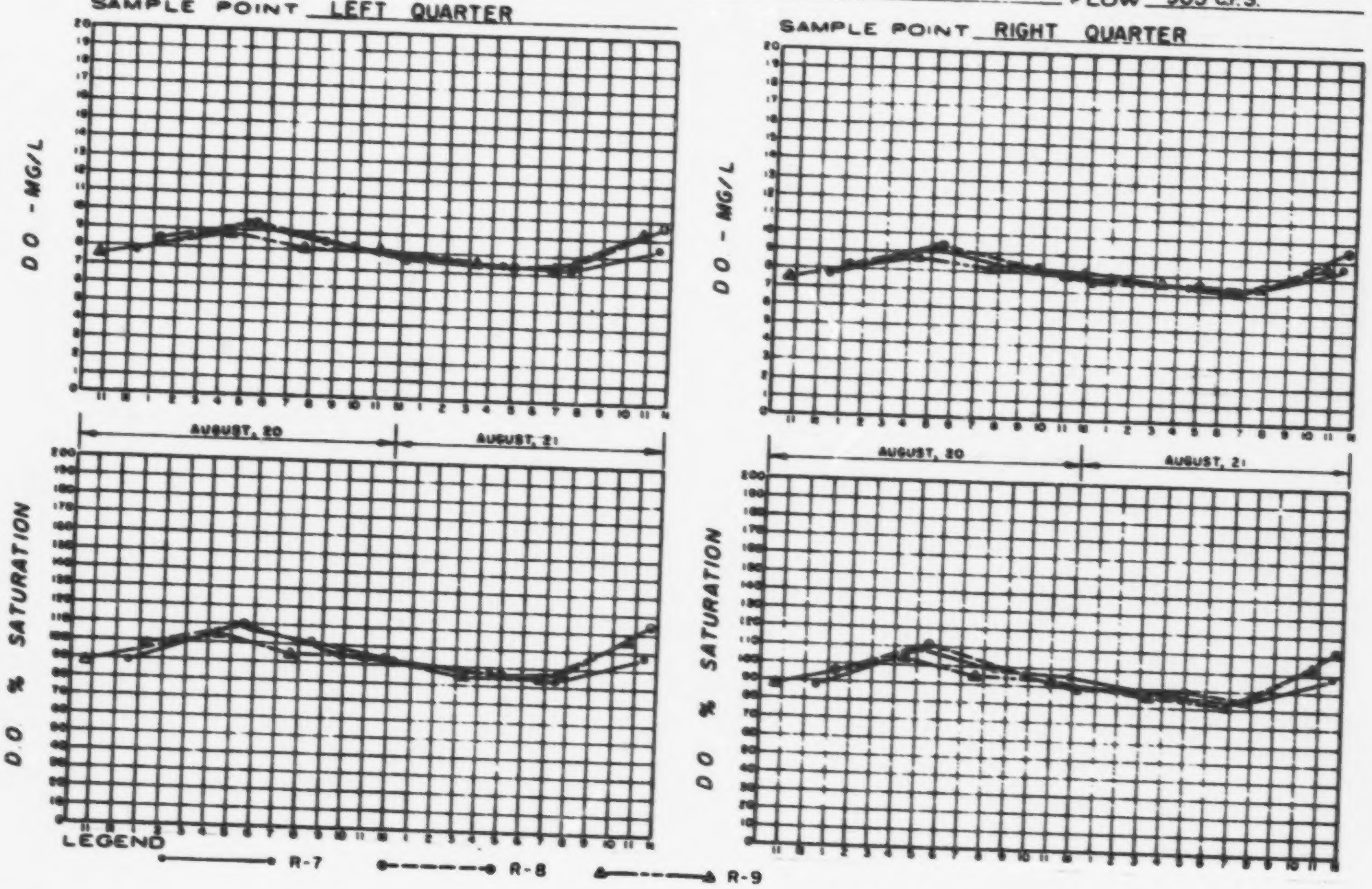


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FIGURE 68 CON'T

DISSOLVED OXYGEN - DIURNAL PATTERNS

STREAM RACCOON RIVER

DATE OCTOBER (16-17) 1969 FLOW 413 C.F.S.

SAMPLE POINT LEFT QUARTER

SAMPLE POINT RIGHT QUARTER

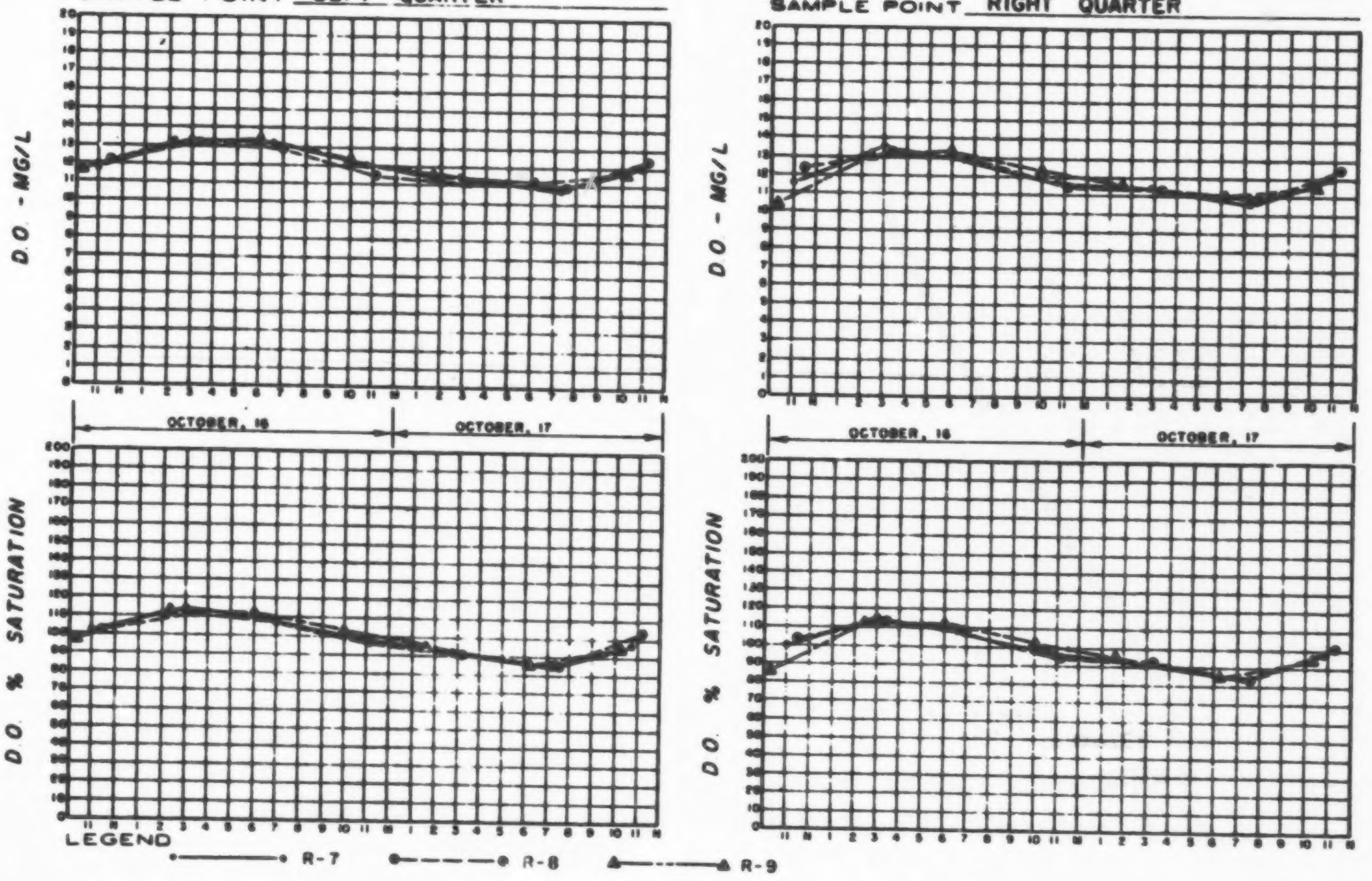
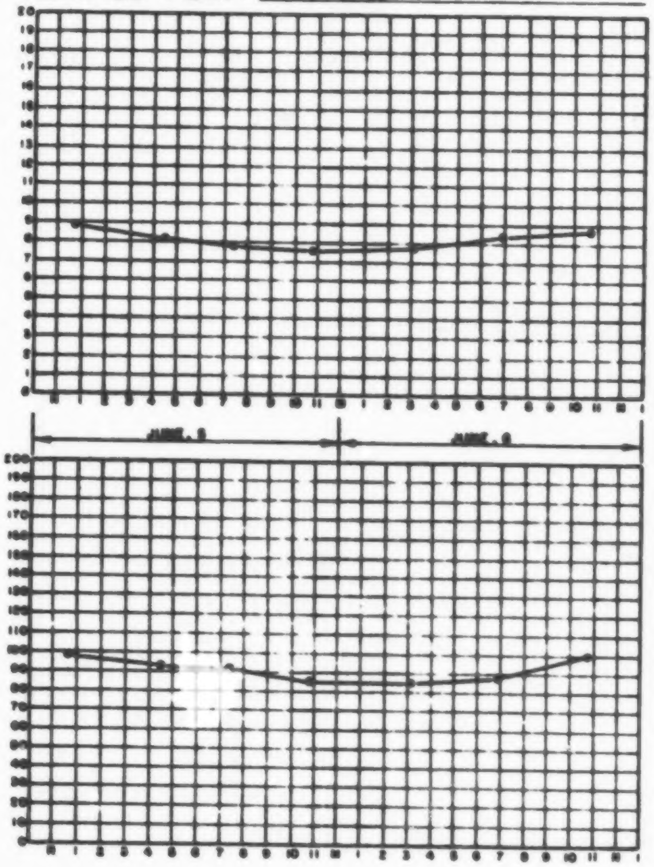


FIGURE 68 CON'T

DISSOLVED OXYGEN - DIURNAL PATTERNS

STREAM WALNUT CREEK (R-10)
DATE JUNE (5-6) 1969 FLOW
SAMPLE POINT MIDPOINT



STREAM FOURMILE CREEK (R-14)
FLOW
SAMPLE POINT MIDPOINT

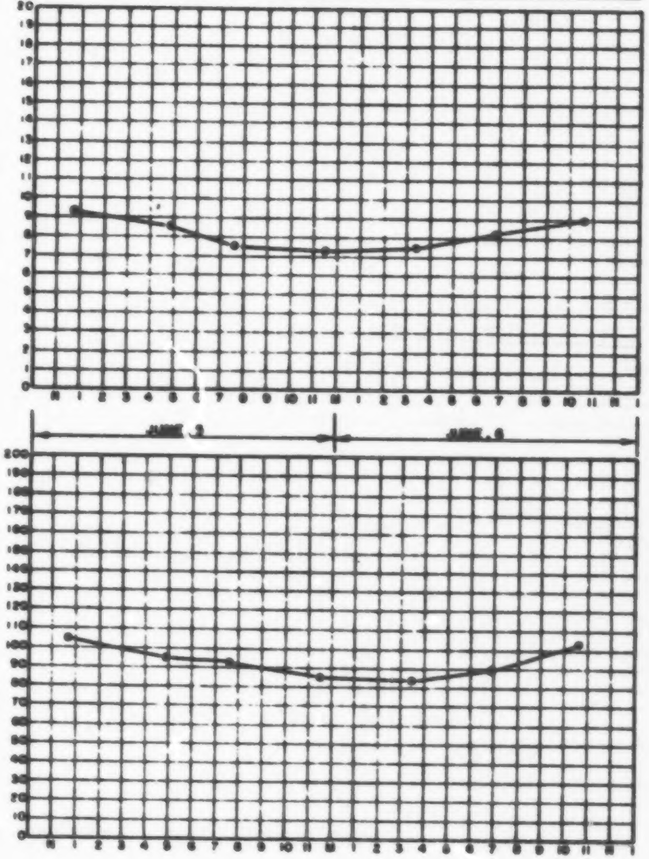
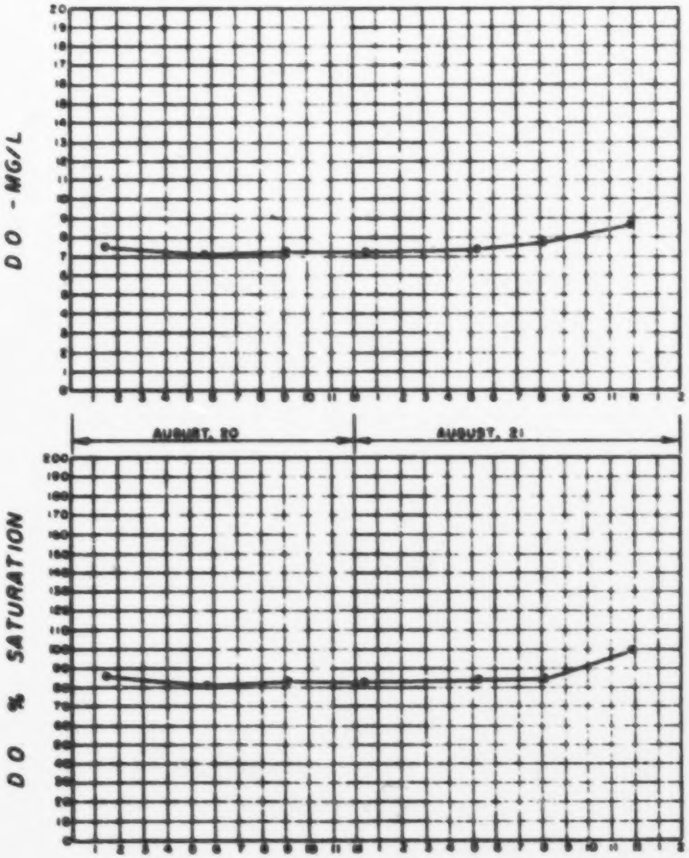


FIGURE 69

DISSOLVED OXYGEN - DIURNAL PATTERNS

STREAM WALNUT CREEK (R-10)
DATE AUGUST (20-21) 1969 FLOW 13.25 C.F.S.
SAMPLE POINT MIDPOINT



STREAM FOURMILE CREEK (R-14)
FLOW 17.85 C.F.S.
SAMPLE POINT MIDPOINT

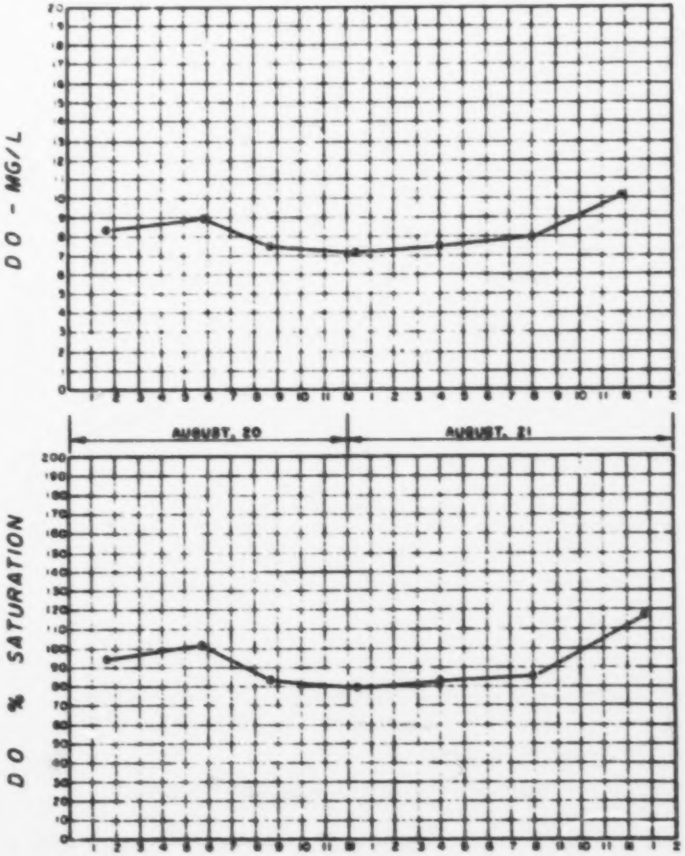


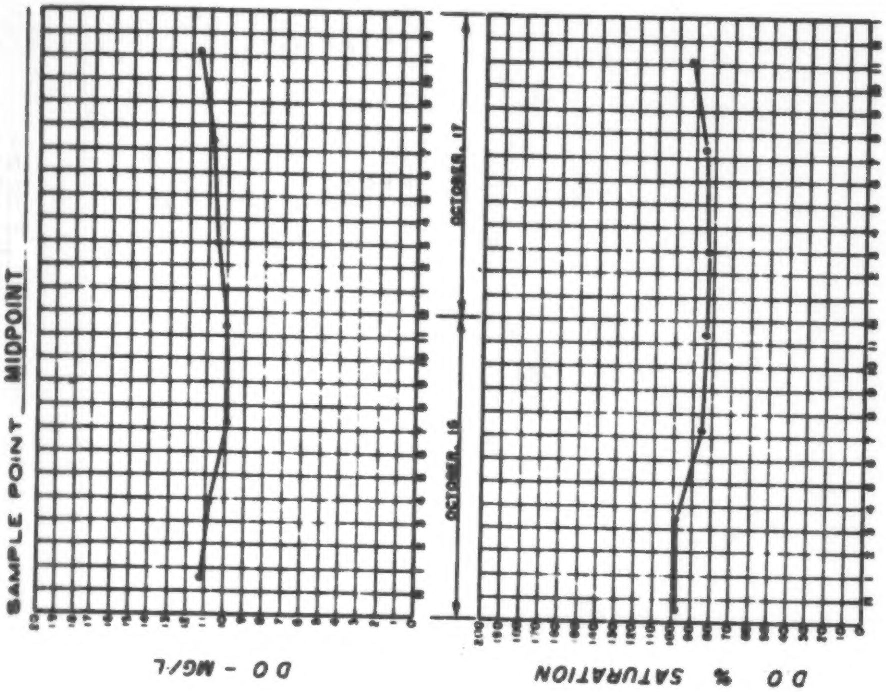
FIGURE 69 CON'T

DISSOLVED OXYGEN - DIURNAL PATTERNS

STREAM FOURMILE CREEK (R-14)

FLOW 14.21 C.F.S.

SAMPLE POINT MIDPOINT



STREAM WALNUT CREEK (R-10)

DATE OCTOBER (16-17) 1969 FLOW 7.25 C.F.S.

SAMPLE POINT MIDPOINT

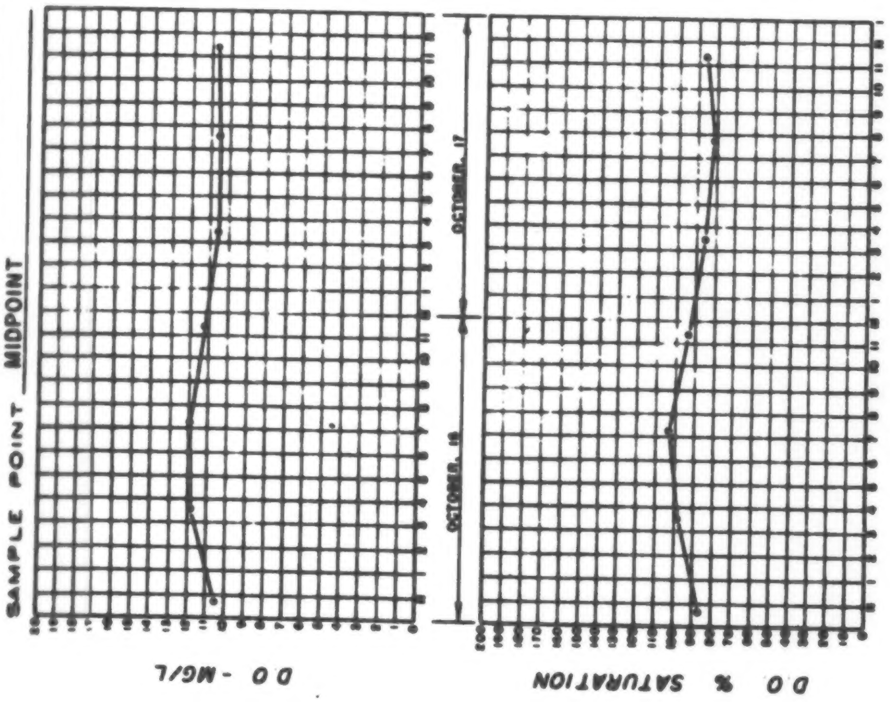


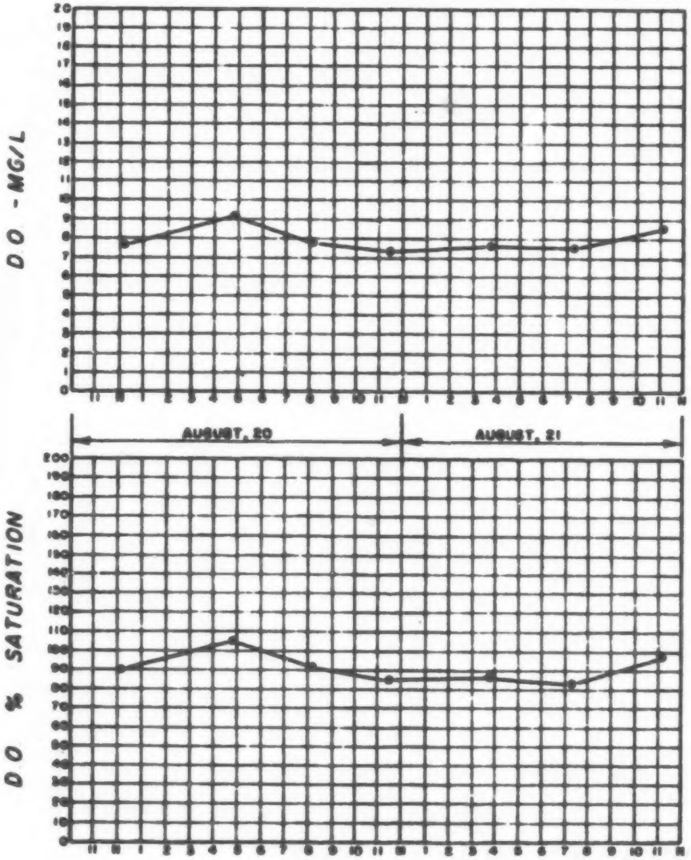
FIGURE 69 CON'T

DISSOLVED OXYGEN - DIURNAL PATTERNS

STREAM BEAVER CREEK (R-15)

DATE AUGUST (20-21) 1969 FLOW 55.49 C.F.S.

SAMPLE POINT MIDPOINT



STREAM SAYLOR CREEK (R-16)

FLOW 5.09 C.F.S.

SAMPLE POINT MIDPOINT

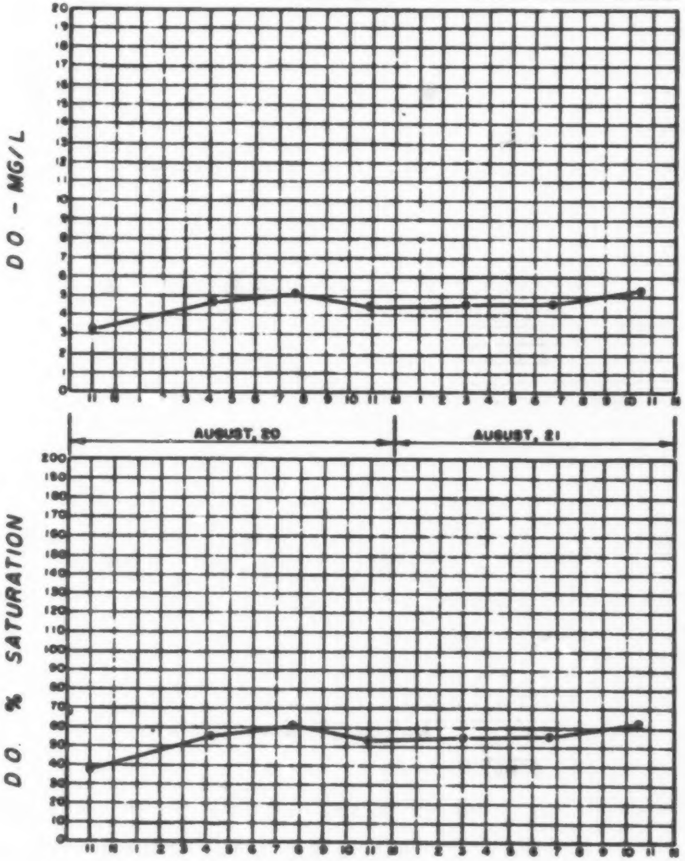
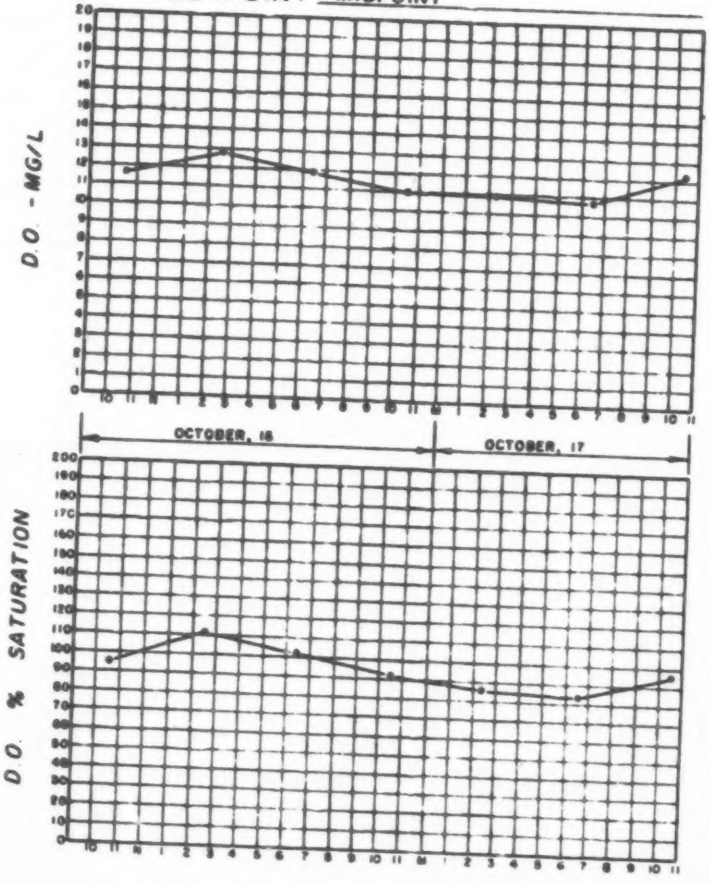


FIGURE 70

DISSOLVED OXYGEN - DIURNAL PATTERNS

STREAM BEAVER CREEK (R-15)
DATE OCTOBER (16-17) 1969 FLOW 48.49 C.F.S.
SAMPLE POINT MIDPOINT



STREAM SAYLOR CREEK (R-16)
FLOW 3.59 C.F.S.
SAMPLE POINT MIDPOINT

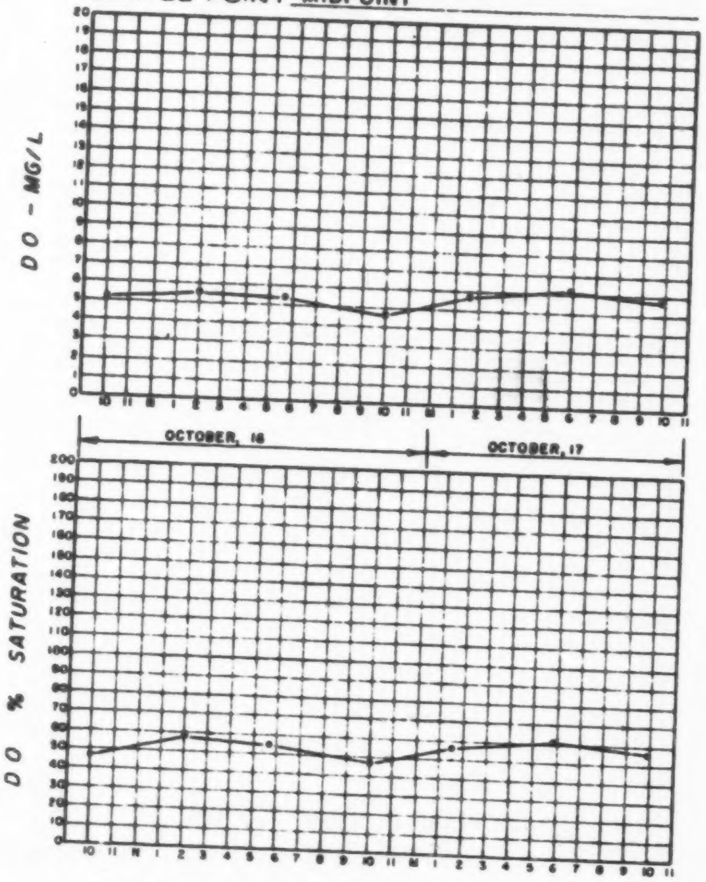


FIGURE 70 CON'T

RIVER STATION R-2 DES MOINES RIVER AT FISHER BRIDGE

RIVER STATION DATA
TABLE 26

RIVER STATION R-2 DES MOINES RIVER AT FISHER BRIDGE

TABLE 26 CON'T

RIVER STATION R-2: DES MOINES RIVER AT FISHER BRIDGE

DATE	BY	RIVER FLOW (CFS)	pH	[illegible]	DO	BOD	COD	NITROGEN [illegible]	NITROGEN [illegible]	NITROGEN [illegible]	NITROGEN [illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	TOTAL SOLIDS TOTAL	TOTAL SOLIDS VOLATILE	TOTAL SOLIDS FIXED	SUSPENDED SOLIDS TOTAL	SUSPENDED SOLIDS VOLATILE	SUSPENDED SOLIDS FIXED	DISSOLVED SOLIDS TOTAL	DISSOLVED SOLIDS VOLATILE	DISSOLVED SOLIDS FIXED	[illegible]	PLANKTON	[illegible]	[illegible]	PESTICIDES	PESTICIDES	PESTICIDES	TOTAL COLIFORMS	FECAL COLIFORMS
6-27-60	ERI	7340			7.3	5.7		1.61	0.21	0.05	8.8		0.6								12																		
7-3-60	ERI	18,300			6.6	3.7		0.77	0.38	0.05	8.6		0.5								60																		
7-9-60	ERI	18,600			7.6	9.6			0.49	0.02	9.5		0.3																										
7-18-60	ERI	12,600			5.8	10.7		0.07	0.47	0.01	4.9		0.9								16.3																		
7-20-60	ERI	7100			7.0	5.6		0.05	0.27	0.09	5.6		0.9								50																		
7-28-60	ERI	8,700			7.1	6.6		0.47	0.20	0.01	6.1		0.5								8																		
8-7-60	ERI	4210			3.6	6.1		0.34	0.49	0.01	5.3		0.4								56																		
8-14-60	ERI	2,520			8.2	5.7		2.6	0.8	0.01	2.8		0.0								59																		
8-20-60	MDP	1300			6.8	11.9	38.6		0.79	0.01	0.68	0.79	0.8	89.5	268	6	88.3	141	120	294	14	486	178	308	102	18	84	402	105	297	0		5	<.005					
8-22-60	ERI	2,730			7.6	8.0		1.30	0.52	0.01	1.90		0.0								96																		
8-16-60	MDP	590			11.97	8.3	58.2		0.56	0.001	0.05	0.26	0.04	187	287	15	18.2	48.9	510	29.9	18	487	130	357	57	22	35	375	62	303	0.0	107,150	19	<.005					

RIVER STATION R-3: DES MOINES RIVER AT EUCLID AVENUE

3-6-60	SHL	170	8.7	2	17.0	14	[illegible]	28	T	.0002	T	2.9	1.0											
3-12-60	SHL	250	8.8	1	19.7	7		190	0.01	.0006	.01	1.5	0.8											
3-15-60	SHL	285	8.7	6	16.9	13			.001				0.8											
3-18-60	SHL	310	8.7	10	11.6	19		200	T	.0001	T	1.7	0.3											
3-22-60	SHL	242			15.9	18			T				0.5											
3-29-60	SHL	212	8.8	10	9.6	19		17	T	.0001	T	1.7	0.3											
4-2-60	SHL	206	8.6	4	9.0	17		20	0.01	.0002	T	1.5	0.3											
4-6-60	SHL	301	8.6	6	8.5	17		19	T	.0001	T	1.5	0.4										430	
4-10-60	SHL	176	8.6	9	9.2	30		19	T	.0002	T	1.5	0.3										4600	
4-12-60	SHL	220	8.6	12	6.7	20		20	0.08	.0001	0.1				550	218	352	100	42	58	650	176	758		
4-17-60	SHL	256	8.5	19	7.1	18		21	0.01	.0002	0.2	2.1	0.6										26,000	
4-24-60	SHL	1150	7.8	8.0	8.7	12		61	0.27	.0002	2.1	4.8	1.8											47,300
5-1-60	SHL	700	8.6	17	8.6	19		56	0.06	.0003	T	1.5	0.3											390
5-7-60	SHL	380	8.6	12	7.1	19		26	0.07	.0002	T	1.2	0.2											80
5-14-60	SHL	580	8.7	160	5.5	14	40.2	17	T	.0001	T	1.3	0.6	110	460	160	520	70	27	65	990	113	277		
5-21-60	SHL	252	8.6	130	9.1	14		16	T	.0002	T	1.0	0.3	110											780
5-28-60	SHL	260	8.5	150	9.1	11		16	T	.0002	T	1.0	0.3	55											660
6-4-60	SHL	270	8.0	26	6.7	7		16	0.01	.0001	T	1.0	0.3	66											7,800
6-12-60	SHL	520	7.6	210	5.4	12		69	0.31	.033	3.3	6.8	1.2	2400											98,000
6-19-60	SHL	920	8.2	23	6.6	6		19	0.07	0.26	5.6	1.3	0.6	988	672	195	677	279	35	248	595	160	239		2600
6-25-60	SHL	1460	8.1	360	5.5	11		67	0.25	0.12	3.4	3.9	0.4	2400	910	960	2570	958	170	1616	324	170	156		22,000
7-2-60	SHL	9,200	8.0	22	7.1	9		21	0.12	0.080	9.6			660	888	260	638	656	98	856	654	162	272		6,300
7-9-60	SHL	1480	8.6	260	9.4	8		22	T	.0040	6.1	1.3	0.4	260	656	758	648	202	58	464	658	200	258		500
7-16-60	SHL	730	8.2	25	5.5	15		53	0.12	.0080	1.8	0.9	0.2	160	679	282	279	160	39	101	935	163	272		2,300
7-25-60	SHL	1080	8.6	27	6.9	9		18	0.06	.008	6.1	1.2	0.7	190	955	304	352	179	19	110	624	182	392		30
7-30-60	SHL	1400	8.3	70	9.8	8		22	0.05	0.035	3.8	1.0	0.3	100											500
8-6-60	SHL	1100	8.8	38	16.8	15		22	0.08	.0057	3.8	0.8	0.3	100											1,100
8-13-60	SHL	752	8.4		7.5	19		21	0.05	.0004	0.2	0.9	0.2	66											1,080
8-24-60	SHL	610	7.8	27	5.8	4		15	T	.0002	0.2	0.7	0.2	42											46
8-27-60	SHL	555	8.2	19	8.1	4		15	0.05	.0003	0.1	0.6	0.2	60											2,400
9-3-60	SHL	260	8.2	21	7.0	6		15	0.01	.0002	T	0.7	0.2	[illegible]											220
9-10-60	SHL	315			8.7	8		14	T	.0009	0.1	0.8	0.9	85											500
9-17-60	SHL	220	7.8	20	5.6	8		13	T	.0002	T	0.7	0.2	82											220
9-24-60	SHL	580	8.2	20	7.2	12		18	T	.0002	T	1.0	0.2	94											180

TABLE 26 CON'T

RIVER STATION R-3 DES MOINES RIVER AT EUCLID AVENUE

DATE	BY	RIVER FLOW (CFS)
11-5-68	SHL	4620
12-3-68	SHL	1980
1-7-69	SHL	800
1-14-69	HDR	580
1-20-69	SHL	640
2-6-69	HDR	510
2-19-69	SHL	650
2-22-69	HDR	490
2-25-69	HDR	410
2-26-69	HDR	630
2-28-69	HDR	510
3-13-69	SHL	620
4-[illegible]-69	SHL	15,900
4-23-69	HDR	16,600
6-5-69	HDR	3080
7-8-69	SHL	10,700
8-11-69	SHL	1100
8-20-69	HDR	1580
9-16-69	SHL	250
9-30-69	SHL	572
10-16-69	HDR	594

RIVER STATION R-4 DES MOINES RIVER AT ELM STREET

2-6-69	HDR	510
2-22-69	HDR	480
2-25-69	HDR	410
2-26-69	HDR	630
2-28-69	HDR	543
4-23-69	HDR	16,600
6-5-69	HDR	3080
8-20-69	HDR	1580
10-16-69	HDR	594

TABLE 26 CON'T

RIVER STATION R-5 DES MOINES RIVER AT S.E. 14TH STREET

DATE	BY	RIVER FLOW (C.F.S.)	pH	WATER TEMP °C	DO	BOD	COD	NITROGEN				PHOSPHATES		TOTAL COLIFORMS PER 100 ML	FECAL COLIFORMS PER 100 ML
3-6-68	SWL	297	[illegible]	1.5	10.1	18	[illegible]	20	T	[illegible]	03	12	05		
3-12-68	SWL	621	7.6	2	12.7	8		15	001	[illegible]	06	12	07		
3-15-68	SWL	529	[illegible]	7	13.6	10			T				06		
3-19-68	SWL	516	8.1	10	12.7	13		19	T	[illegible]	01	14	06		
3-22-68	SWL	500			13.7	16			T				08		
3-26-68	SWL	380			12.8	13		10	001	[illegible]	08	09	08		
3-29-68	SWL	[illegible]	8.7	11	10.7	10		18	T	[illegible]	T	10	03		
4-2-68	SWL	[illegible]	8.4	9	9.9	12		14	001	[illegible]	01	18	06		
4-6-68	SWL	404	7.8	6	9.9	14		18	T	[illegible]	T	16	06		
4-10-68	SWL	364	[illegible]	10	10.7	15		20	[illegible]	[illegible]	T	16	03		
4-12-68	SWL	303	8.0	11	7.8	16		17	[illegible]	[illegible]	01			10,000	
4-17-68	SWL	396	8.5	14	9.3	17		20	001	[illegible]	01	20	07	59,000	
4-24-68	SWL	2500	7.7	7	9.6	9		26	014	[illegible]	09	21	08		
5-1-68	SWL	1960	8.7	17	[illegible]	14		32	[illegible]	[illegible]	08	15	02	240,000	
5-7-68	SWL	[illegible]	[illegible]	19	7.8	18		23	[illegible]	[illegible]	T	01	02		20,000
5-14-68	SWL	672	7.9	19	7.2	11		17	T	[illegible]	01	12	04		2,100
5-21-68	SWL	529	[illegible]	14	9.3	10		16	[illegible]	[illegible]	T	11	02		9,100
5-28-68	SWL	530	8.1	15	9.4	8		10	T	[illegible]	T	09	03		
6-4-68	SWL	[illegible]	[illegible]	27	7.8	8		16	011	[illegible]	01	07	04		3,900
6-12-68	SWL	1810	7.9	23	5.9	7		16	[illegible]	011	08	13	05		3,900
6-19-68	SWL	1880	8.0	25	6.8	5		17	[illegible]	[illegible]	05	11	03		6,800
6-25-68	SWL	[illegible]	8.1	25	6.6	5		21	[illegible]	09	30	16	06		80,000
7-2-68	SWL	3900			7.5	6		28	018	011	60				19,000
7-9-68	SWL	2130	8.0	[illegible]	8.0	7		25	[illegible]	[illegible]	39	06	15		10,000
7-16-68	SWL	1190	[illegible]	25	6.6	10		26	[illegible]	[illegible]	18	12	20		6,500
7-23-68	SWL	[illegible]	[illegible]	[illegible]	7.3	8		16	005	[illegible]	66	16	08		
7-30-68	SWL	1460	[illegible]	[illegible]	[illegible]	10		22	[illegible]	[illegible]	21	08	02		3,100
8-6-68	SWL	1030	[illegible]	24	12.5	9		20	[illegible]	[illegible]	38	09	03		4,600
8-13-68	SWL	[illegible]	8.8	27	11.3	19		23	[illegible]	[illegible]	03	09	01		4,000
8-21-68	SWL	630	7.8	26.6	6.0	9		13	027	[illegible]	02	08	09		9,000
8-27-68	SWL	379	7.8	21	8.0	5		13	017	[illegible]	01	07	04		900
9-3-68	SWL	344	[illegible]	22	10.1	7		13	001	[illegible]	01	09	09		2,500
9-10-68	SWL	678			12.1	8		16	001	[illegible]	01	09	09		27,000
9-17-68	SWL	695	[illegible]	19	7.7	7		16	T	[illegible]	T	07	02		1,400
9-24-68	SWL	903	8.4	20	9.8	8		10	T	[illegible]	02	10	08		4,400
12-3-68	SWL	3970			13.9	3	148	[illegible]	[illegible]	[illegible]	64	06	09		14,000
1-7-69	SWL	1180	7.8	0	12.3	7	[illegible]	15	030	[illegible]	38	12	03		4,000
1-16-69	[illegible]	[illegible]		0											1,800
1-30-69	SWL	1450	7.5	0	10.1	3	[illegible]	18	[illegible]	[illegible]	38	11	11		1,600
2-6-69	[illegible]	[illegible]	7.1		[illegible]	[illegible]	[illegible]		[illegible]	[illegible]	[illegible]	[illegible]	[illegible]		
2-19-69	SWL	[illegible]	7.5	2	[illegible]	3	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	11	11	70	7,100
2-25-69	[illegible]	[illegible]							[illegible]	[illegible]	[illegible]				89
2-26-69	[illegible]	[illegible]				72			271	[illegible]	300				1,000
2-28-69	[illegible]	[illegible]							[illegible]	[illegible]	300				
3-13-69	SWL	1120	7.7	0.8	10.2	3	290	11	[illegible]	[illegible]	30	16	12		
4-1-69	SWL	[illegible]	7.9	2	12.6	6	[illegible]	16	[illegible]	[illegible]	49	15	09		
4-22-69	[illegible]	[illegible]	7.6	12.5	[illegible]	6.6	236		[illegible]	[illegible]	[illegible]	[illegible]	[illegible]		1,600
6-3-69	[illegible]	4480	[illegible]	[illegible]	[illegible]	6.9	[illegible]		[illegible]	[illegible]	[illegible]	[illegible]	[illegible]		3,100
7-8-69	SWL	21,000			7.2	2	300	13	[illegible]	[illegible]	60	08	05		
8-11-69	SWL	5,230	8.6	[illegible]	7.3	4	612	29	T	[illegible]	66	04	04		5,000
8-20-69	[illegible]	2,000			7.25	4.4	412		026	[illegible]	160	076	[illegible]		86,000

(CONTINUED)

TABLE 26 CON'T

RIVER STATION R-5 DES MOINES RIVER AT SE 14TH STREET

RIVER STATION R-6 DES MOINES RIVER AT HIGHWAY 46

TABLE 26 CON'T

RIVER STATION R-6 DES MOINES RIVER AT HIGHWAY 46

2-24-69	HDR	4,000					
3-13-69	SHL	1,180	7.6	11	10.6	7	[illegible]
4-1-69	SHL	23,000	7.9	1	11.9	6	
4-29-69	HDR	22,000	7.6	15	9.6	[illegible]	18.7
4-29-69	ERI	30,000			9.7	3.0	
5-8-69	ERI	12,500			8.5	3.7	
5-23-69	ERI	8,900			9.8	6.7	
6-5-69	HDR	4,600	[illegible]	[illegible]	11.0	5.0	18.7
6-19-69	ERI	8,650			8.8	5.5	
7-2-69	ERI	27,000			7.8	5.0	
7-8-69	ERI	31,000			7.57	5.6	
7-8-69	SHL	31,000			7.5	8	26.0
7-17-69	ERI	18,800			6.7	4.9	
7-23-69	ERI	12,800			7.3	8.3	
7-29-69	ERI	10,400			7.6	6.9	
8-6-69	ERI	7,040			7.7	4.6	
8-11-69	SHL	5,230	8.5	25.5	6.9	25	14.7
8-15-69	ERI	4,050			8.9	5.5	
8-20-69	HDR	2,800			7.15	11.6	18.5
8-21-69	ERI	2,540			[illegible]	10.0	
9-16-69	SHL	1,740	8.5	20.9	8.7	10.0	[illegible]
9-30-69	SHL	950		21	6.8	14	[illegible]
10-16-69	HDR	1,030		9	11.30	12.7	32.6

RIVER STATION R-7 RACOON RIVER AT I-35 BRIDGE

2-6-69	HDR	330			[illegible]	14.7	19.55
4-23-69	HDR	3,880	7.9	12	[illegible]	6.6	8.3
6-5-69	HDR	1,200			11.5	8.3	21.2
8-20-69	HDR	904			7.8	7.2	27.9
10-16-69	HDR	419		8.5	11.7	9.0	25.6

RIVER STATION R-8 RACOON RIVER AT 63RD STREET

2-6-69	HDR	330	7.6	0	[illegible]	10.8	8.9
4-23-69	HDR	3,880	7.7	12	9.6	6.9	22.2
6-5-69	HDR	1,200			14.0	10.1	
8-20-69	HDR	905			8.2	8.1	29.1
10-16-69	HDR	419		7.5	12.2	7.8	

TABLE 26 CON'T

RIVER STATION R-9 RACOON RIVER AT 5TH STREET

DATE	BY	FLOW (CFS)	pH	WATER TEMP °C	DO	BOD	COD	NITROGEN TOTAL	NH_3	NO_2	NO_3	PHOSPHATES TOTAL	ORTHO	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	TOTAL COLIFORMS MF PER 100 ML			FECAL COLIFORMS MF PER 100 ML		
4-2-68	SHL	161	8.4	9	9.5	9		17	.00	.0008	T	.12	.03							25																							
4-10-68	SHL	128	8.8	11	9.5	10																																	990			490	
5-1-68	SHL	[illegible]	8.5	17	8.2	7		21	.008	.011	.41	.10	.05							25																							
5-14-68	SHL	260	8.1	18	5.9	7	[illegible]	15	T	.0025	T	.10	.04								950	462	179	325	190	52	127	288	93	96													
6-4-68	SHL	200	8.0	27	7.7	7		18	.001	.0004	T	.08	.02								14																					2,000	
7-2-68	SHL	2120	8.0	22	7.0	9		37	.007	.010	.79										2600																					600	
7-30-68	SHL	750	8.8	25	7.6	10		19	.001	.0008	.02	.08	.02								80																					1,000	
9-10-68	SHL	766			12.9	11		25	T	.0004	T	.11	.03								190																					600	
11-5-68	SHL	1,220		6	10.8	2	126	0.58	.001	.0026	7.0	.07	.06								10	602			8			366														2,100	
12-9-68	SHL	970			12.6	9	229	0.80	.015	.0057	8.9	.06	.04	96	407			[illegible]			85																					1,000	
1-7-69	SHL	640	7.8	.25	10.1	9	121	.047	.047	.010	.78	.08	.06	990	468					10	30	552	272	980	40	12	28	512	280	928								29	8		8	5	
2-6-69	HDR	380	7.8		9.05	15.15	15.36		1.54	.067	6.77	0.112	.0087	223.3	459	7	46.9	160	39.8	54.2	275	638	752	666	286	78	120	366	580	367	393	[illegible]		7								700	
2-19-69	SHL	306	7.8	1	9.0	2	141	0.60	.088	.008	6.5	.10	.08	300	387					26	30																					<100	
3-19-69	SHL	600	7.7	0.3	9.1	9	290	1.3	.26	.0077	3.8	.19	.19	796	510					23	28	484			8			476														900	
4-1-69	SHL	8,770	8.0	1	12.9	6	311	2.1	.064	.010	6.6	.19	.06	64	296					40	640	902			368			336															
4-28-69	HDR	3880		12.8	9.85	6.4	121		.0089	.0021	4.09	.930	.0046	278	439	7	84.3	359	12.5	1.70	56	605	252	420	206	36	170	561	189	576	<.1			[illegible]									
6-5-69	HDR	1,200	6.3	21	11.47	5.5	307		.122	.0066	6.65	.961	.0478	0.0	338	15	250	296.1	11	96	13	652	182	630	44	26	18	561	121	446		[illegible]	<2									6,900	
7-8-69	SHL	5,780			7.2	2	400	19	.008	.0048	8.0	.08	.05	262	356					15	600	750	178	572	292	98	258	458	164	310												32,000	
8-11-69	SHL	1,720	8.6	28.5	6.8	6	996	4.9	T	.0050	5.1	.07	.06	296	560					14	500	1640	326	342	212	122	1080	496	184	252													
8-20-69	HDR	905			7.40	3.4	259		1.95	.001	4.13	.063	.050		310			2272	10.5	15.0	15	782	466	576	98	8	90	605	155	482		[illegible]		<.05								460	
9-16-69	SHL	334	8.3	20.5	8.1	6	266	1.5	T	.0015	2.5	.02	T	220	288					17	10	652	178	278	30	14	96	562	160	222												170	
9-30-69	SHL	339		19	8.5	11	318	1.8	T	.0006	0.1	.02									77	618	150	266	98	36	62	320	114	206													
10-16-69	HDR	615			11.00	7.5	168		.041	0	.046	.043	.005	219	271	5	44.6	46	15.8	17.2	16	350	167	208	98	21	17	327	55	272	0	187,190	0	<.05									

RIVER STATION R-10: WALNUT CREEK AT GRAND AVENUE

6-5-69	HDR			32	8.0	0.5	6.7		9.565	.0061	15	.0026	.0018	1906	580	19	9.6	278.6	45	17.6	12	522	181	581	40	21	19	512	207	536		2,976											
8-20-69	HDR		7.6		7.5	8.7	67.3		0.20	.002	.060	0.58	0.58	181	290	6	46.2	106.9	10.4	19.6	26	626	190	427	254	17	187	324	198	126	0	40,984	16	<.005									
10-16-69	HDR			8	13.5	3.6	16.0		0.26	0.0	.060	.091	.080	306	420	1.2	73.5	264	24.1	30.6	10	695	170	276	40	18	22	482	64	636	0.0	2,100	7	<.005					690			290	

RIVER STATION R-14: 4-MILE CREEK AT SCOTT STREET

6-5-69	HDR		8.64	22	9.2	0.5	14.6		0.135	.0142	0.66	2.68	2.69	1905	439	8	64.6	188.1	29	17.2	9	646	250	386	49	20	23	619	211	485		4,669	<2	<.005									
8-20-69	HDR				13.2	9.9	22.6		1.19	.002	1.66	1.40	2.18	1905	400	5	58.5	20	46.7	77.9	3	600	228	496	67	19	46	640	200	427	0	30,716	8	<.005									
10-16-69	HDR			10	11.2	4.7	21.1		0.62	.002	3.18	6.40	3.50	307	483	0.9	784	280.7	31.5	31.6	32	600	183	313	9	7	2	602	96	588	0.0	6,950	0	<.005					1400			110	

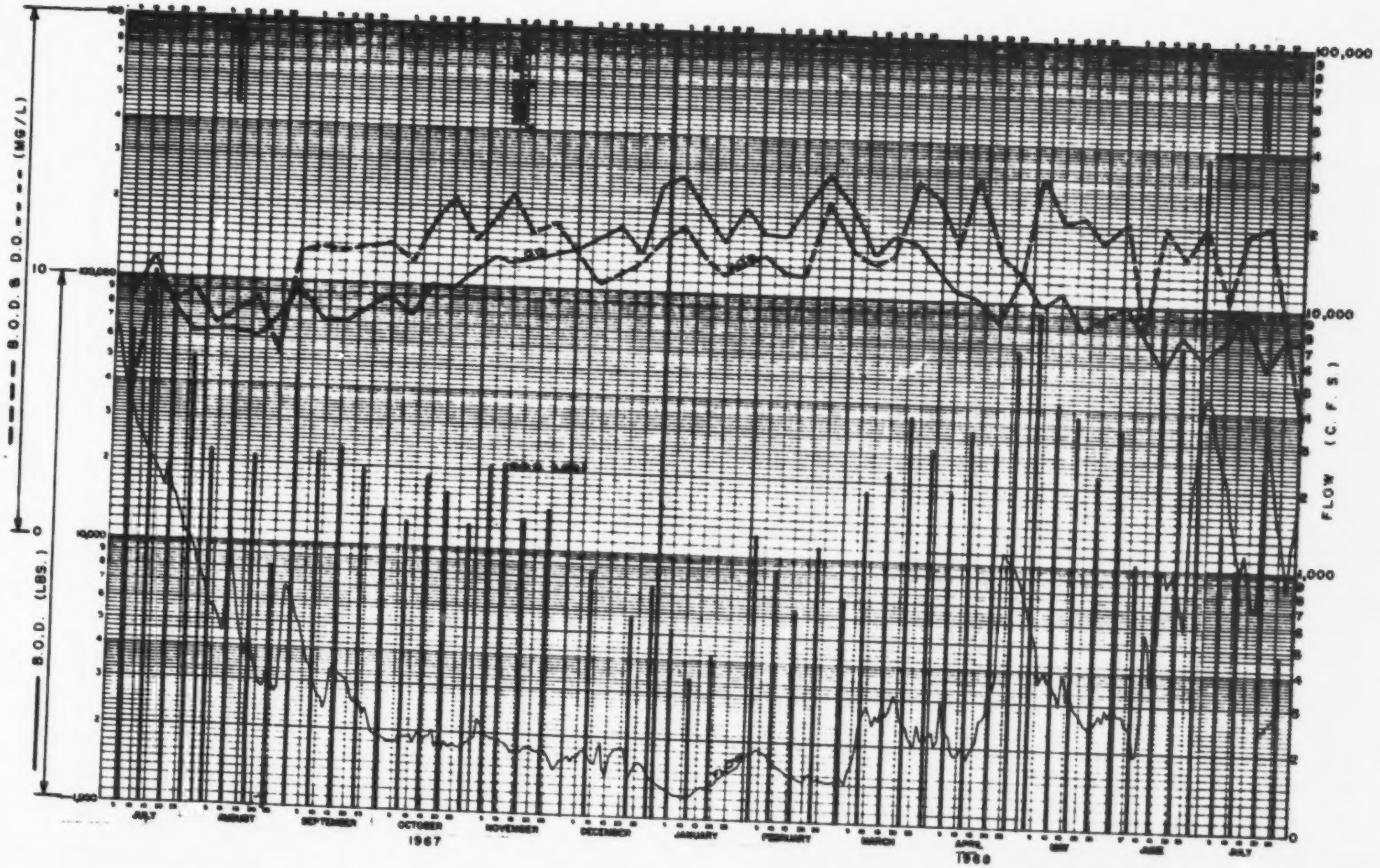
RIVER STATION R-15: BEAVER CREEK AT MERLE HAY ROAD

8-20-69	HDR				7.7	6.5	20.2		.017	.006	3.65	0.36	0.20	1175	500	5	61.9	151.6	15.9	24.6	12	503	205	297	99	20	79	410	165	245	0	26,600	8	<.005									
10-16-69	HDR			7	11.6	9.1	36.4		0.50	.000	2.78	0.66	0.64	276	387	1.2	52.6	275.0	18.9	23.1	6.8	261	61	200	22	7	19	411	80	591	0.0	42,786	0	<.005					1,900			690	

RIVER STATION R-16: SAYLOR CREEK AT 12TH STREET

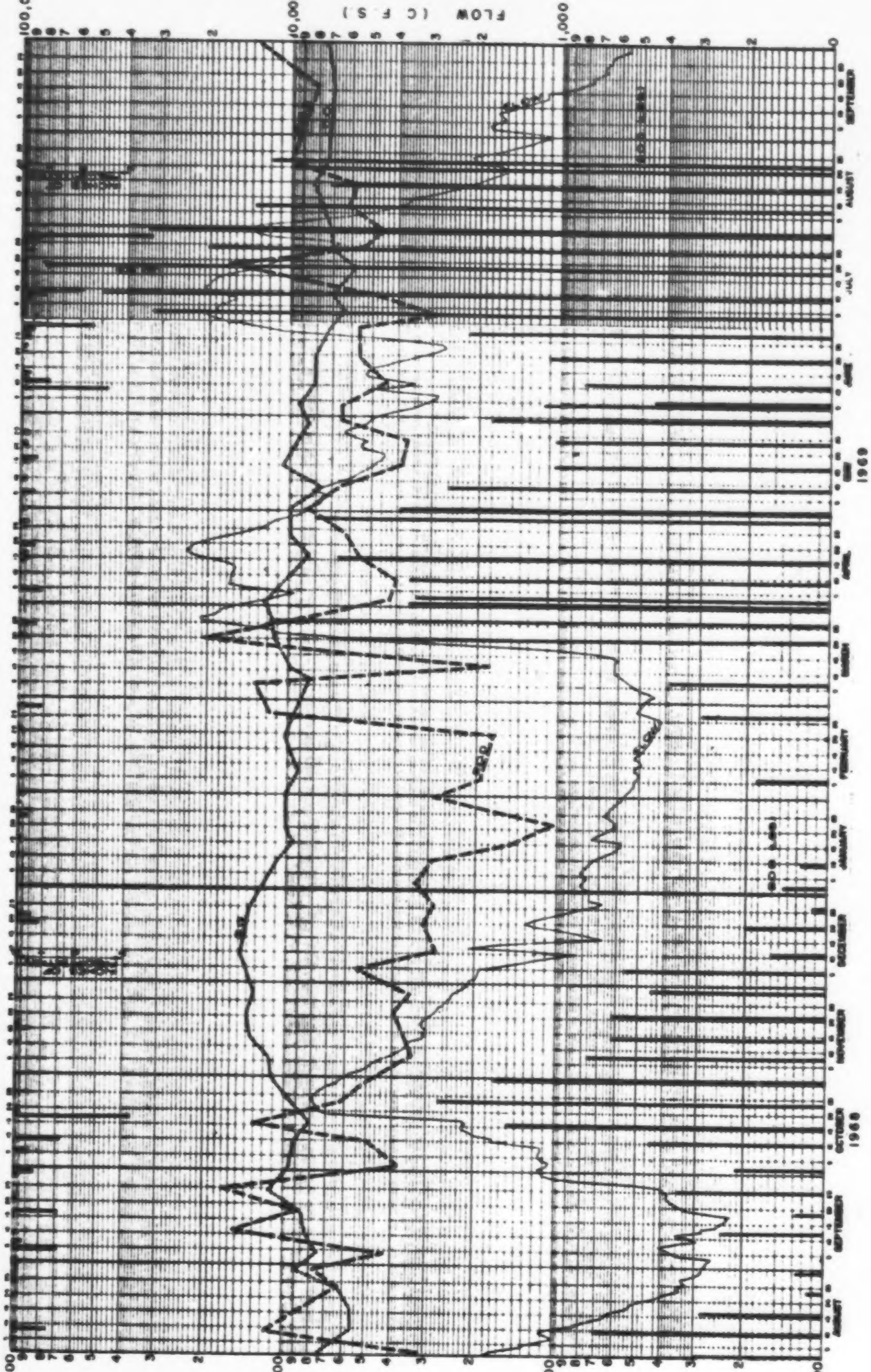
8-20-69	HDR		6.6	24	5.2	19.9	146.0		.095	.027	1.20	1.36	1.90	823	290	9	65.1	15.1	26.6	31.0	86	695	700	495	175	25	190	445	100	290	28	7,960	29	<.005									
10-16-69	HDR			12	6.1	5.7	22.5		.046	.006	.036	1.66	.069	218	280	0.6	58.0	58.6	46.0	31.5	16	581	197	454	85	17	66	460	92	577	0.0	7,715	5	<.005					5400			170	

TABLE 26 CON'T



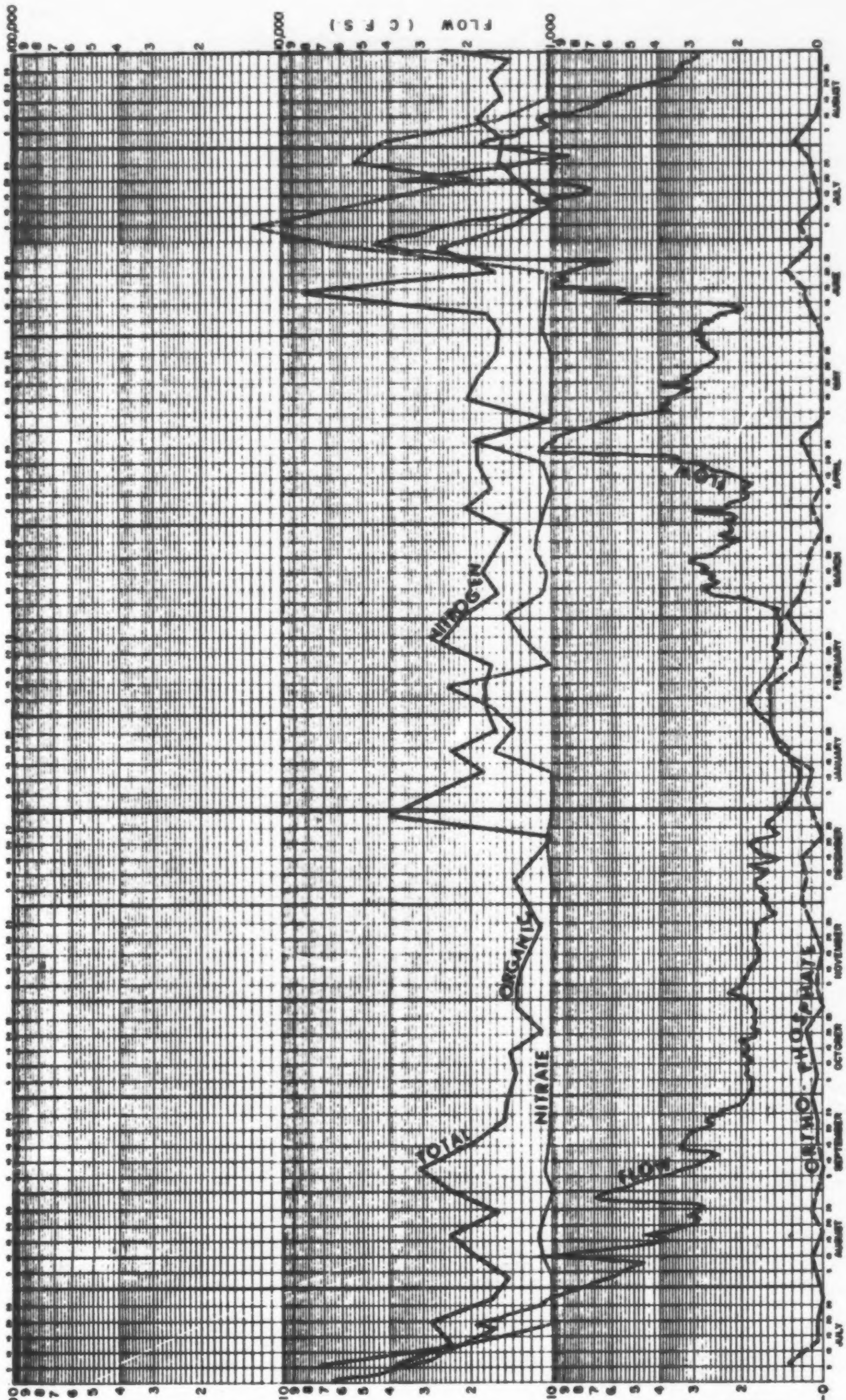
STATION R-2
DES MOINES RIVER-NEAR SAYLORVILLE
B.O.D & D.O. DATA

FIGURE 71



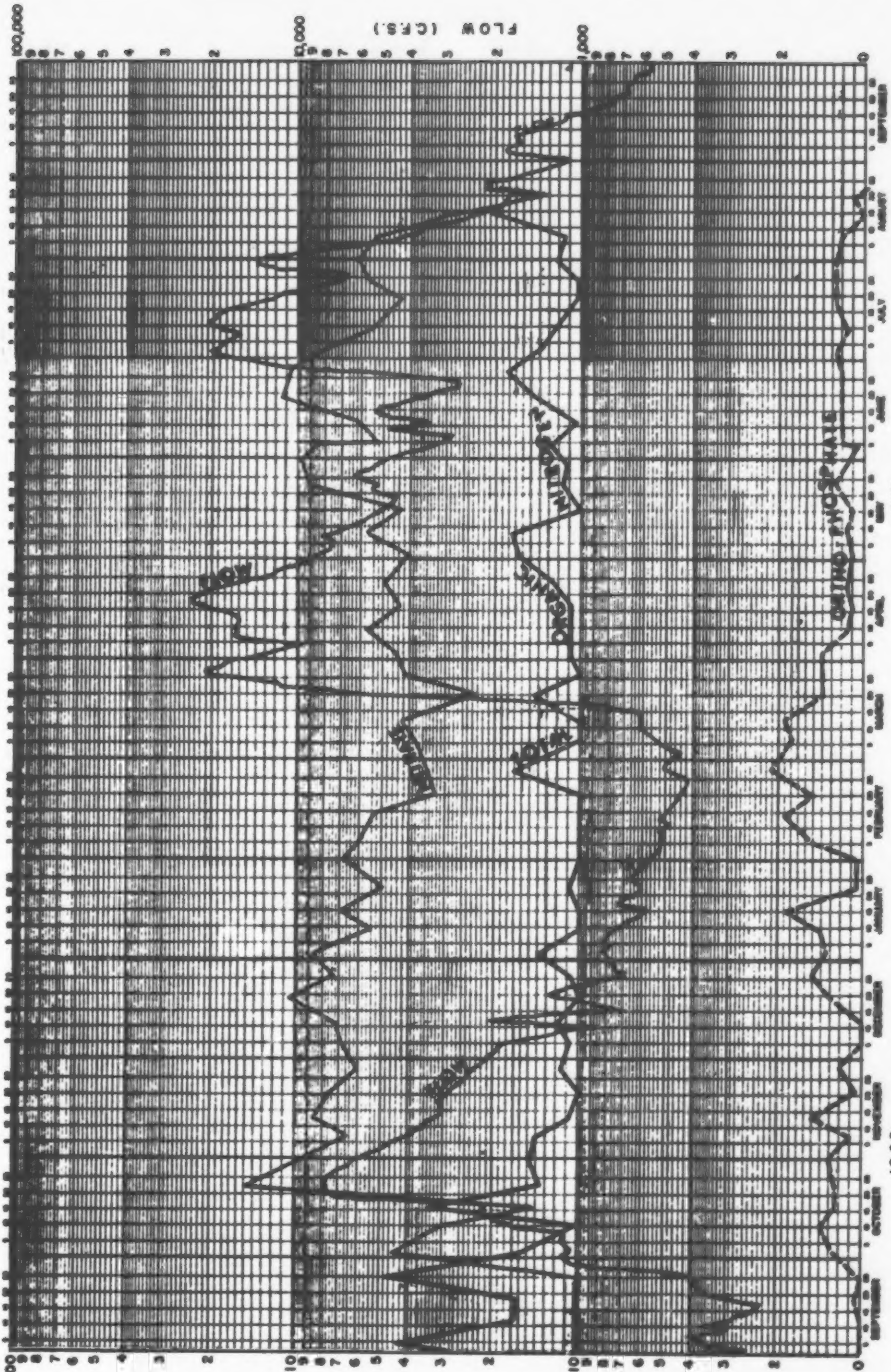
STATION R-2
DES MOINES RIVER - NEAR SAYLORVILLE
B.O.D. & D.O. DATA

FIGURE 71 CON'T



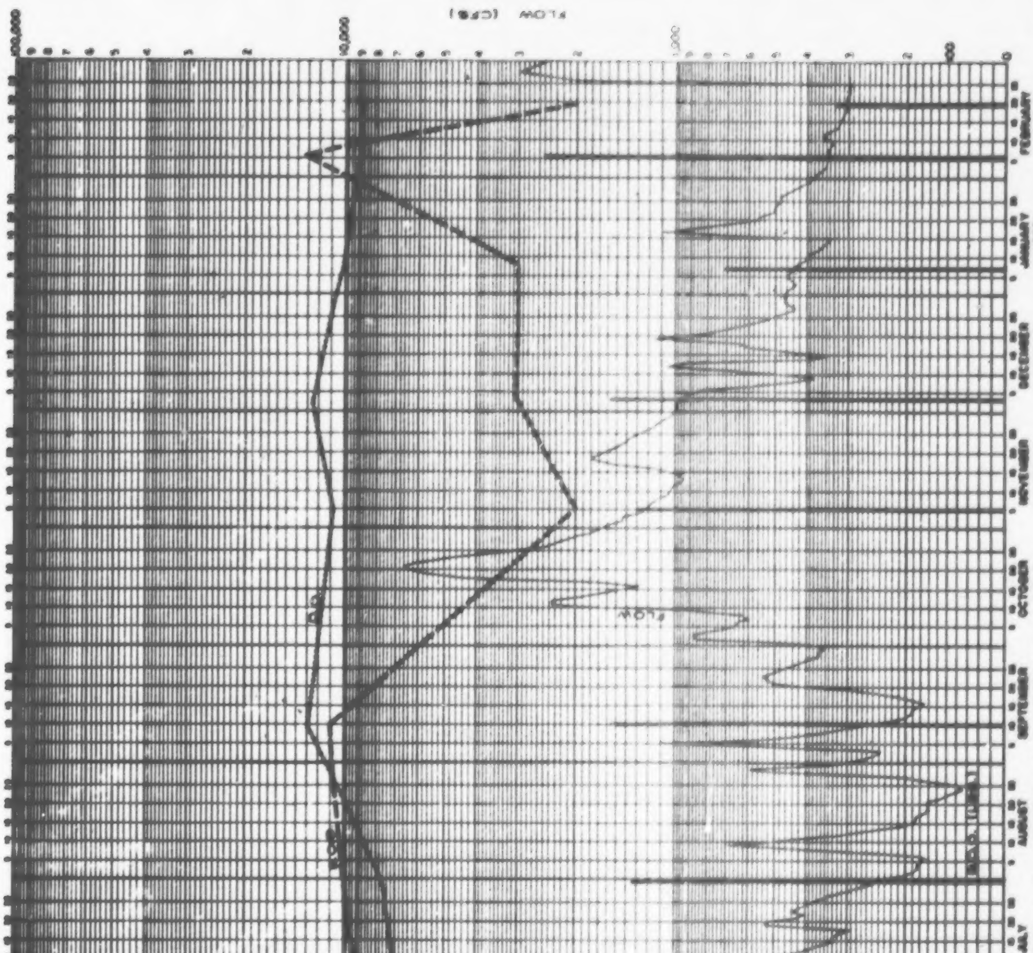
STATION R-2
DES MOINES RIVER - NEAR SAYLORVILLE
PHOSPHATE & NITROGEN DATA

FIGURE 72



STATION R-2
DES MOINES RIVER - NEAR SAYLORVILLE
PHOSPHATE & NITROGEN DATA

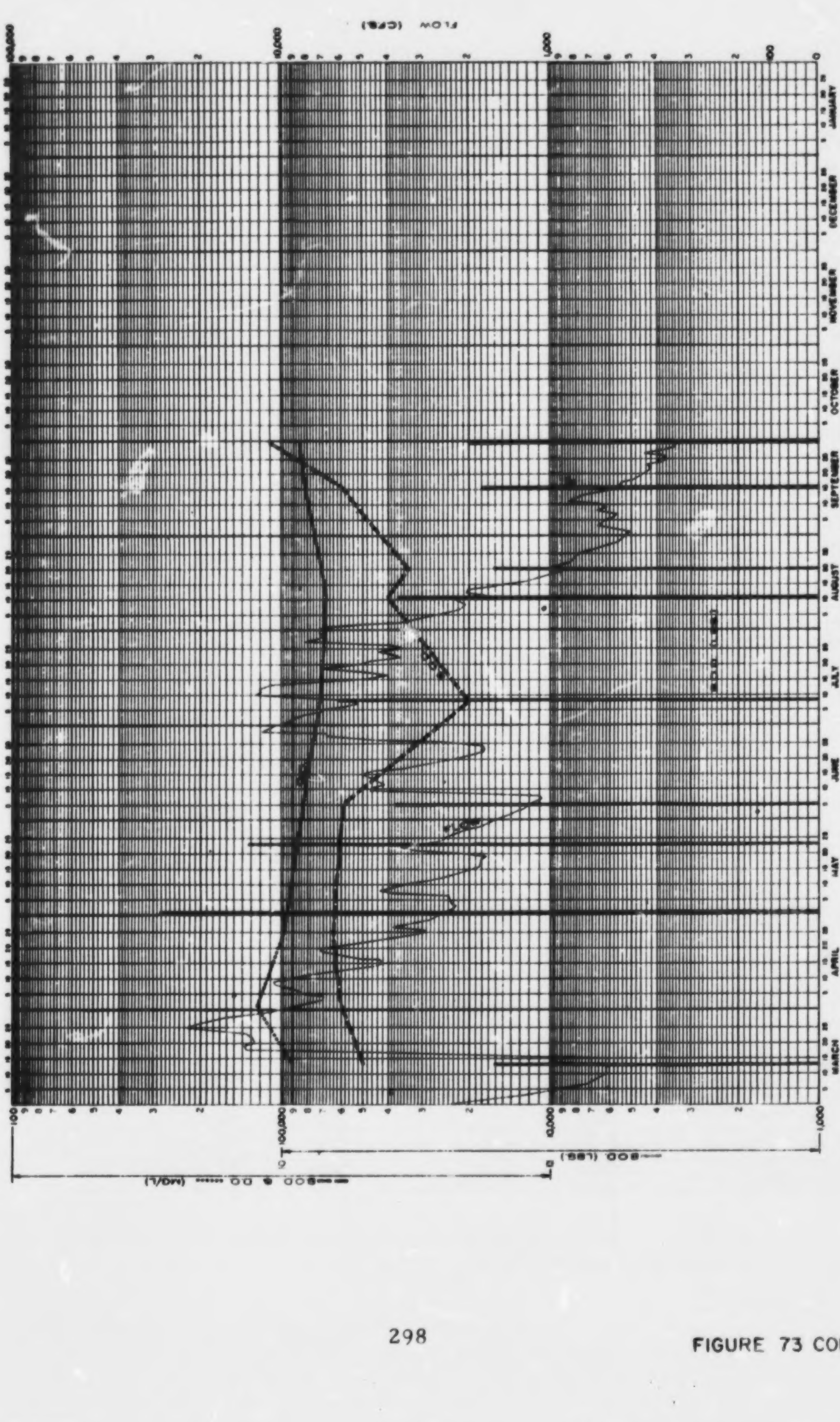
FIGURE 72 CON'T



B.O.D. & D.O. DATA

STATION R-9: RACCOON RIVER AT 5TH STREET

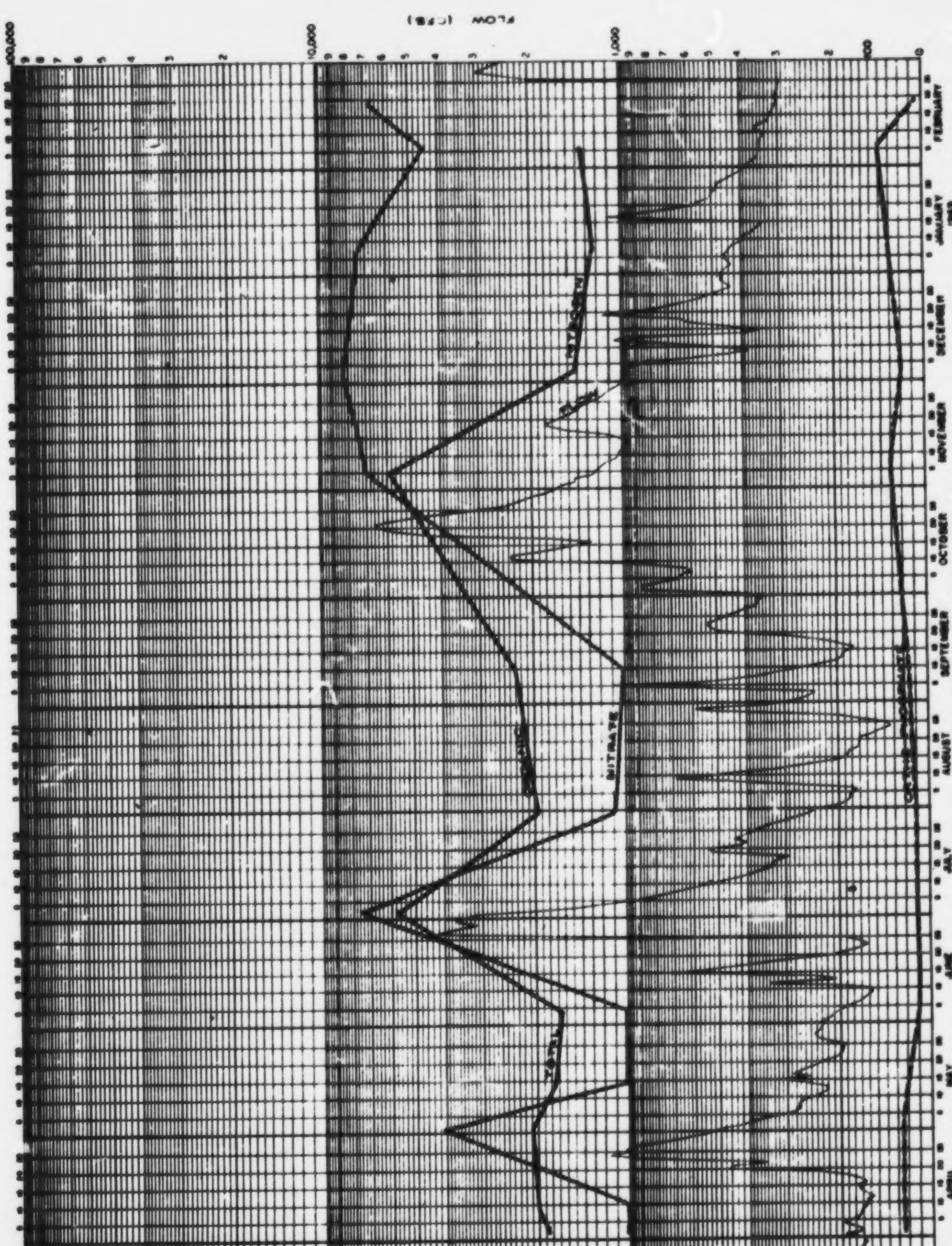
FIGURE 73



B.O.D. & D.O. DATA

STATION R-9: RACCOON RIVER AT 5TH STREET

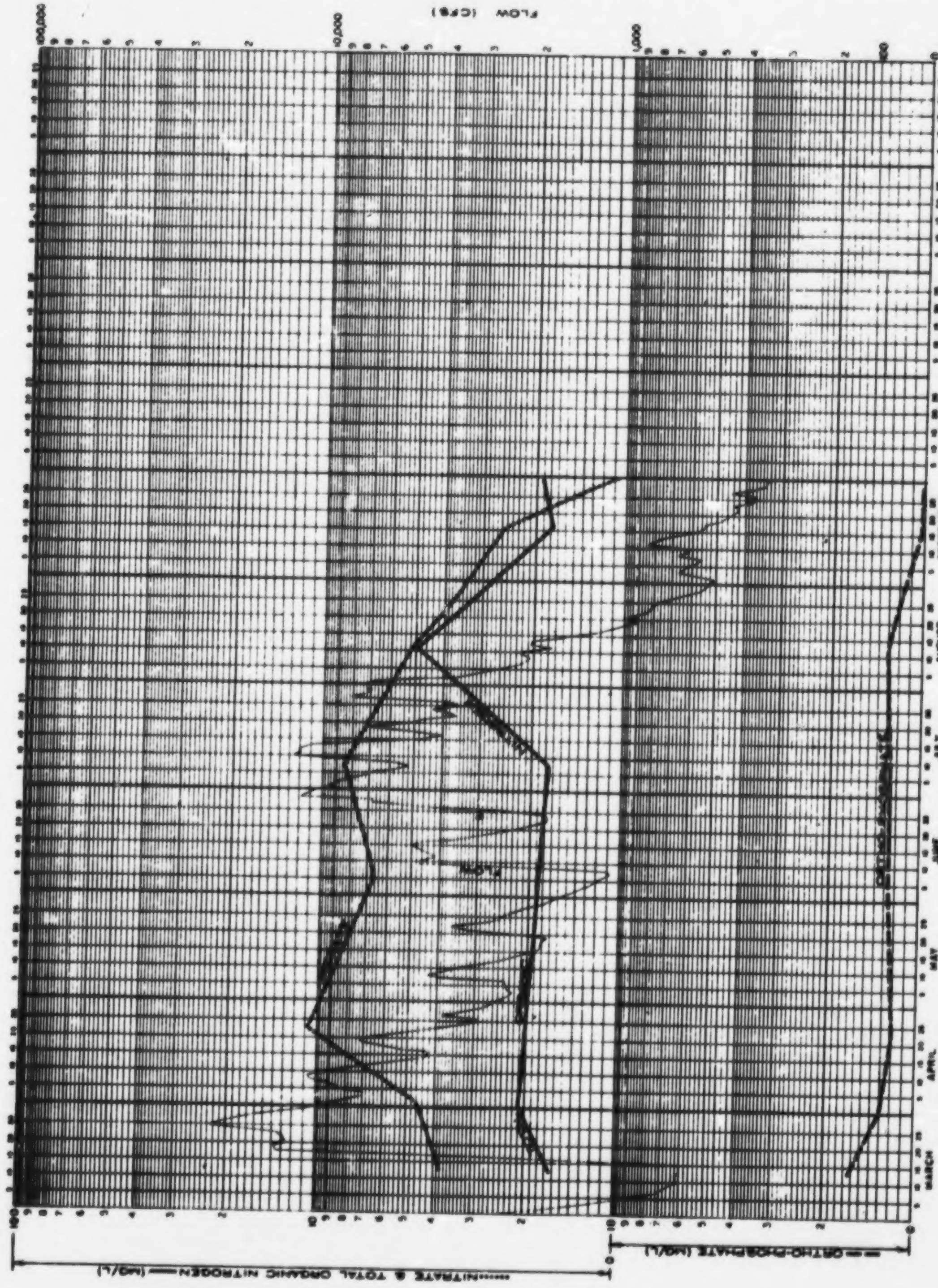
FIGURE 73 CON'T



STATION R-9: RACCOON RIVER AT 5TH STREET

PHOSPHATE & NITROGEN DATA

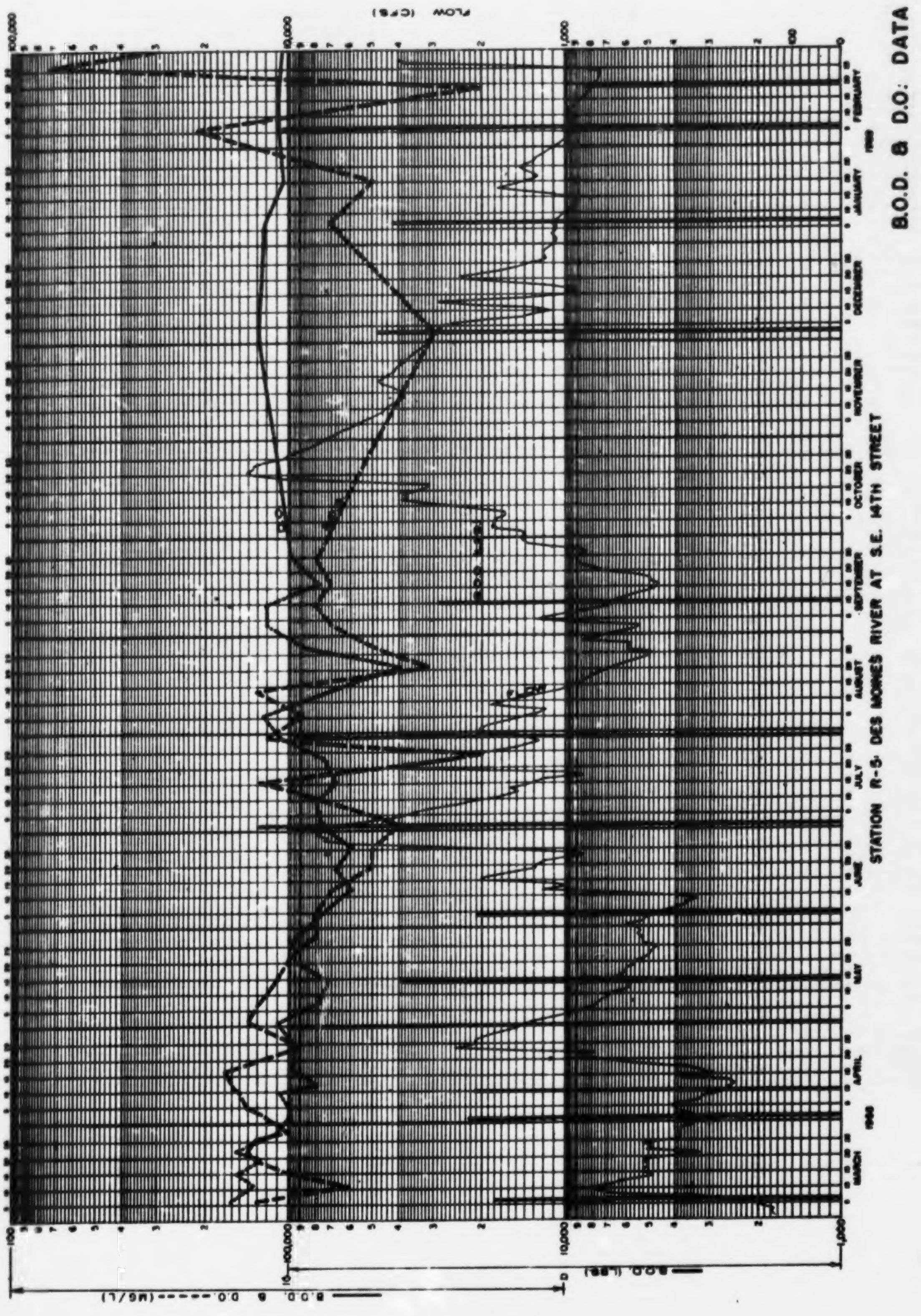
FIGURE 74



STATION R-9: RACCOON RIVER AT 5TH STREET

PHOSPHATE & NITROGEN DATA

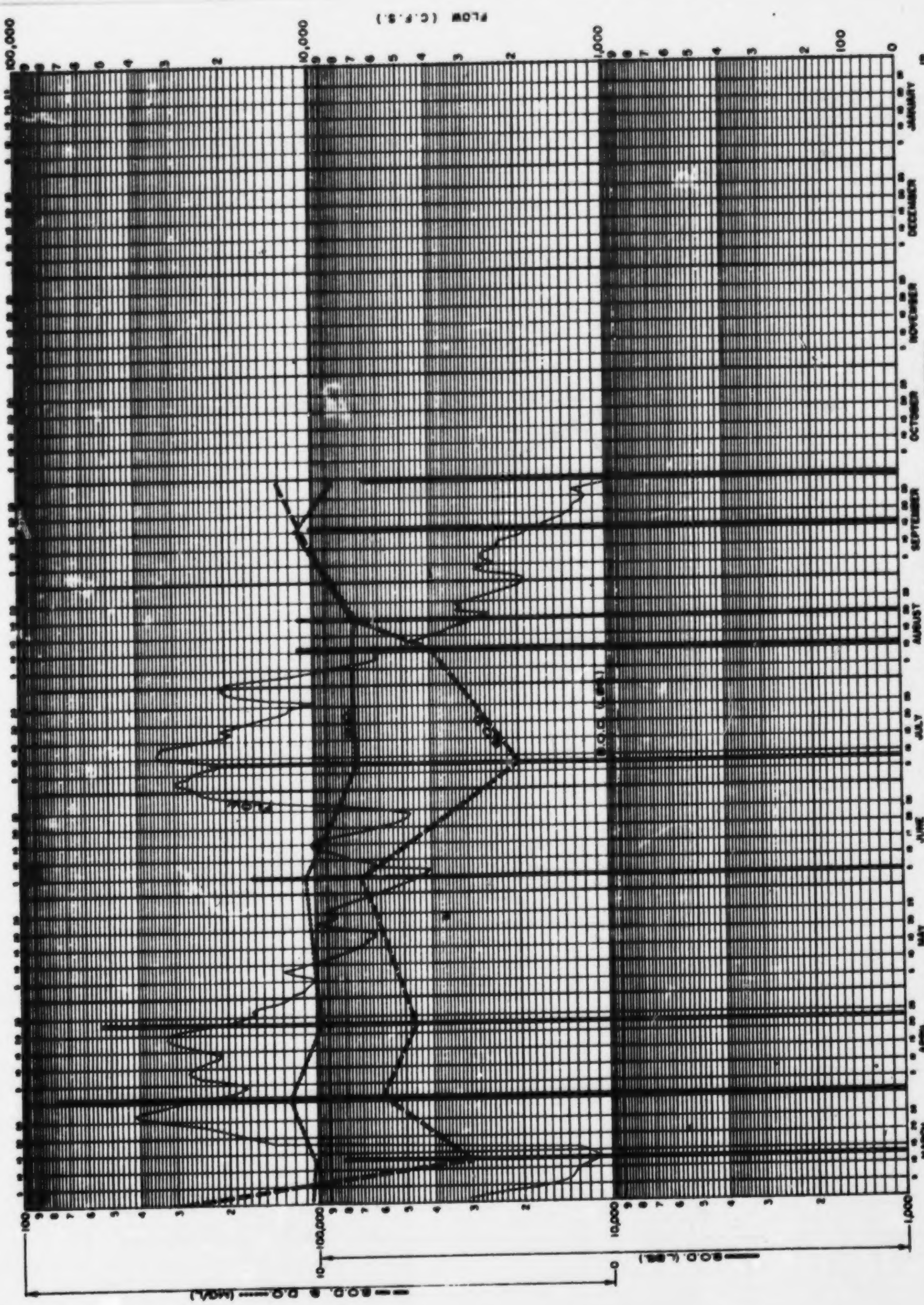
FIGURE 74 CON'T



B.O.D. & D.O.: DATA

STATION R-5 DES MOINES RIVER AT S.E. 14TH STREET

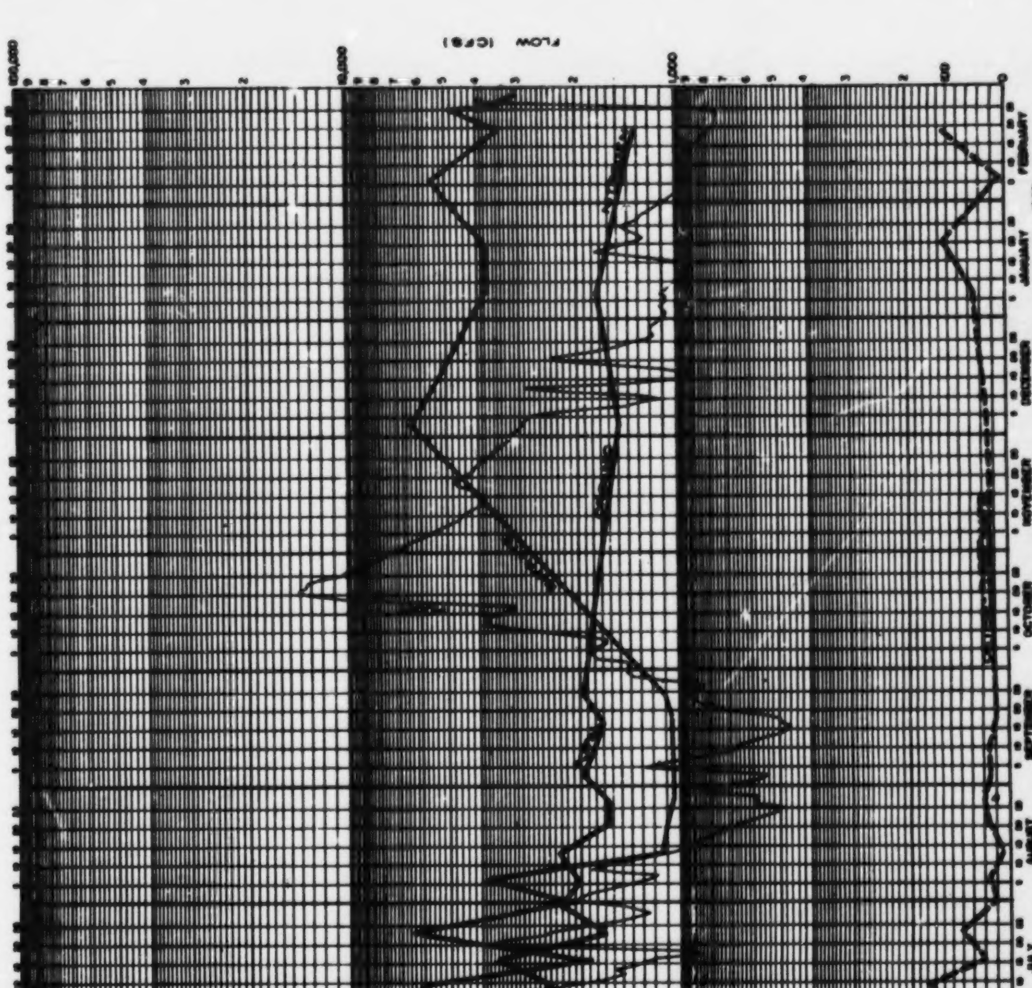
FIGURE 75



STATION R-5: DES MOINES RIVER AT S.E. 14TH STREET

B.O.D. & D.O. DATA

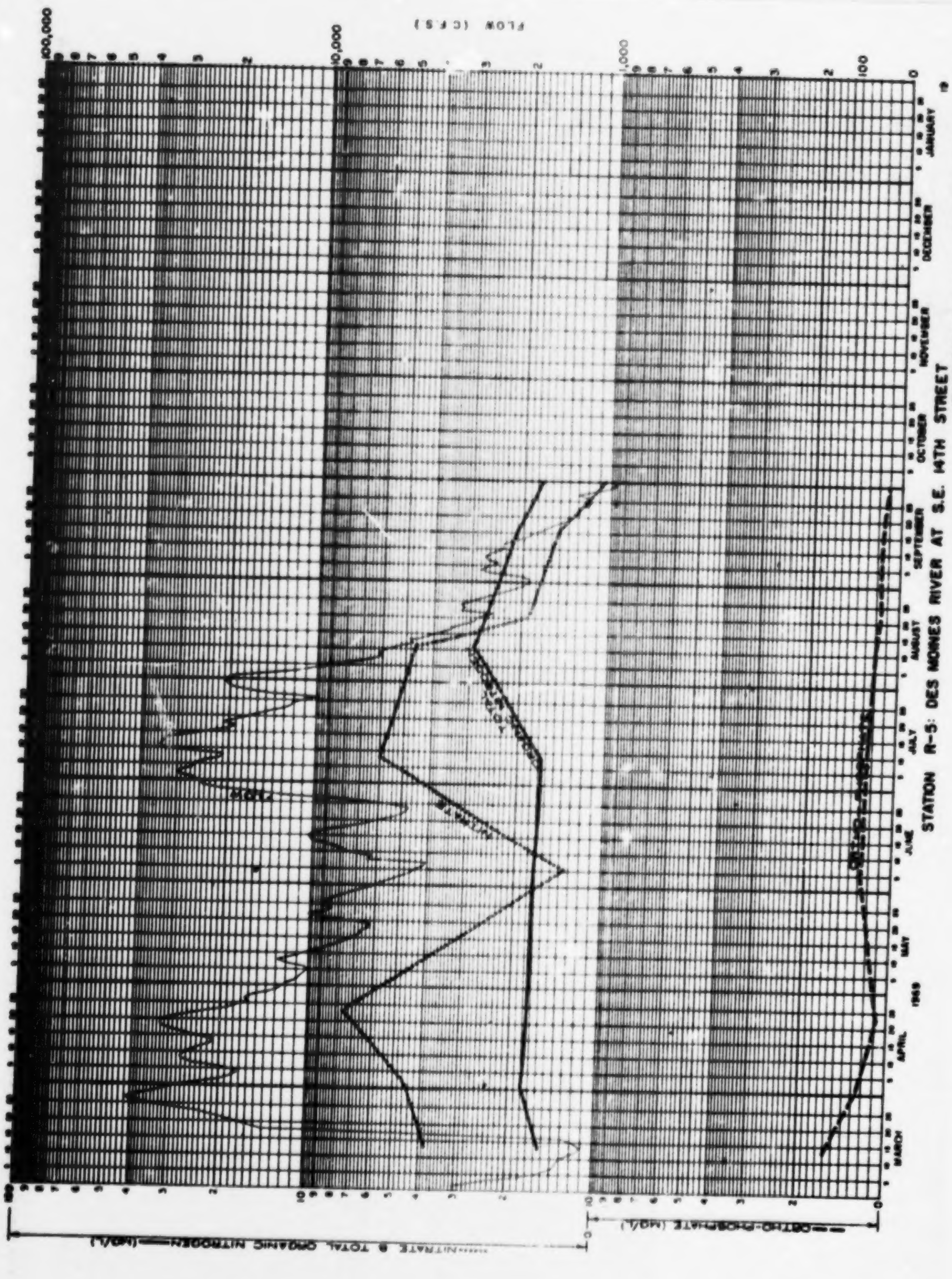
FIGURE 75 CON'T



STATION R-5: DES MOINES RIVER AT S.E. 14TH STREET

PHOSPHATE & NITROGEN DATA

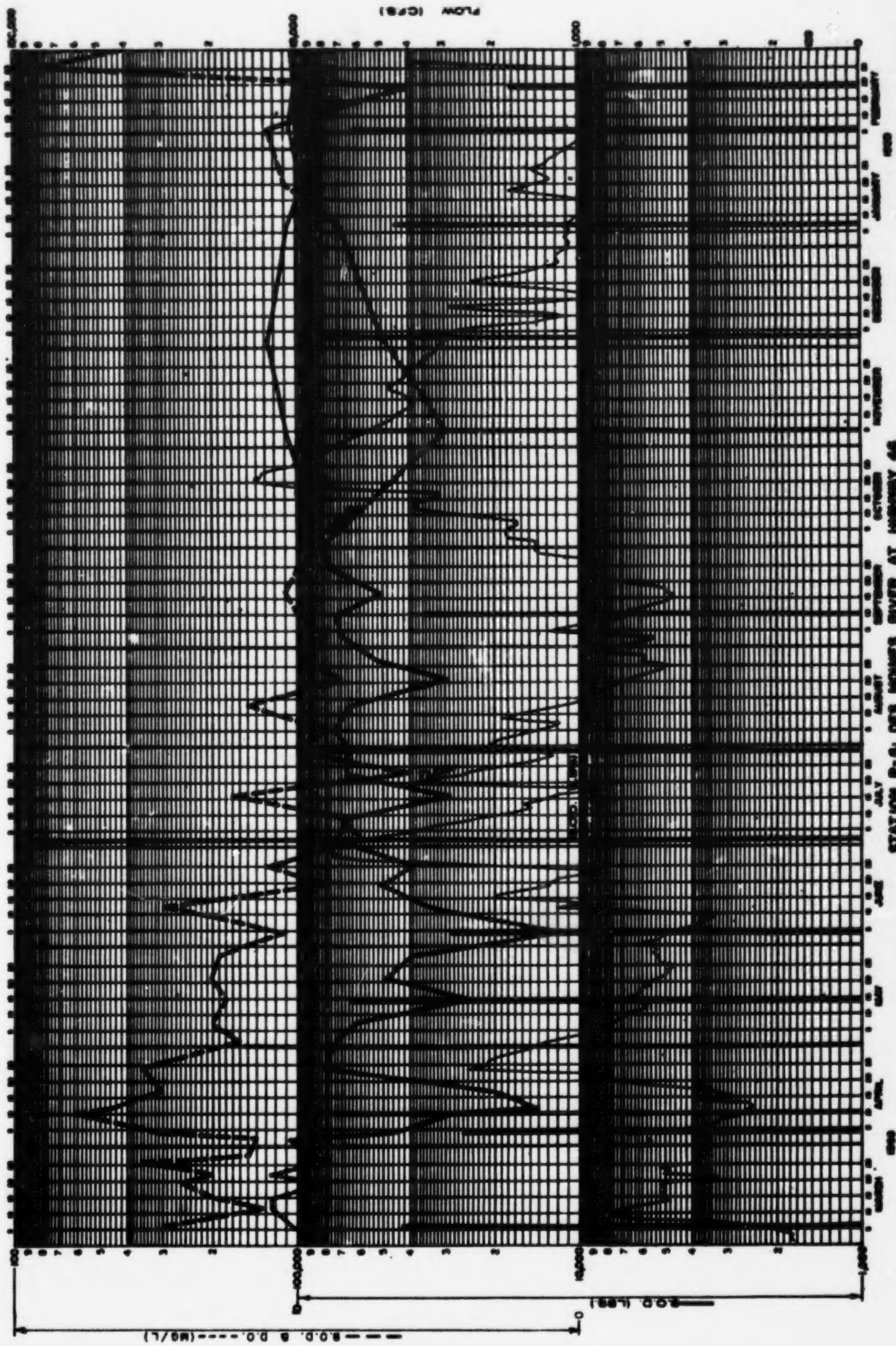
FIGURE 76



STATION R-5: DES MOINES RIVER AT S.E. 14TH STREET

PHOSPHATE & NITROGEN DATA

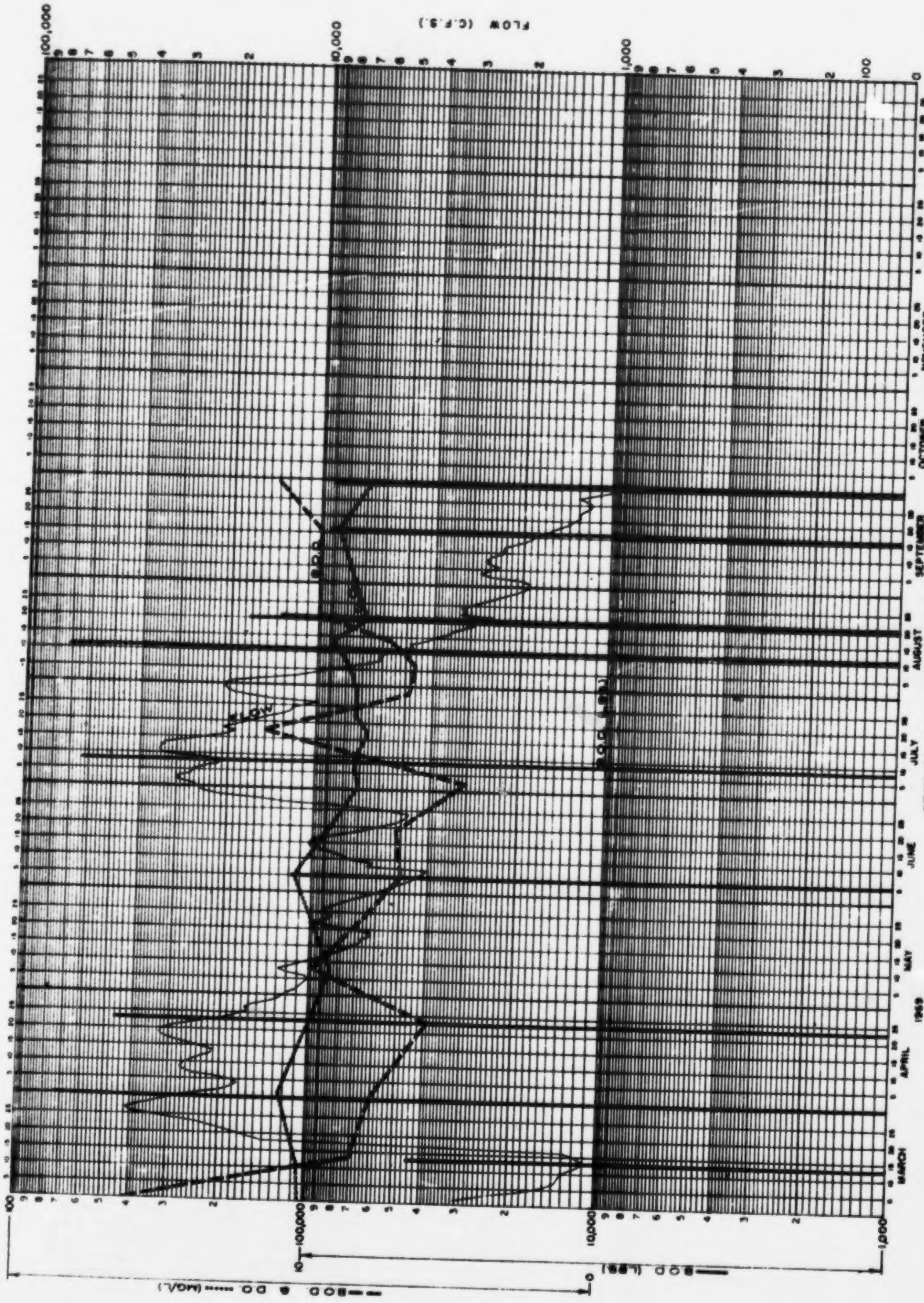
FIGURE 76 CON'T



STATION R-9: DES MOINES RIVER AT HIGHWAY 46

B.O.D. & D.O. DATA

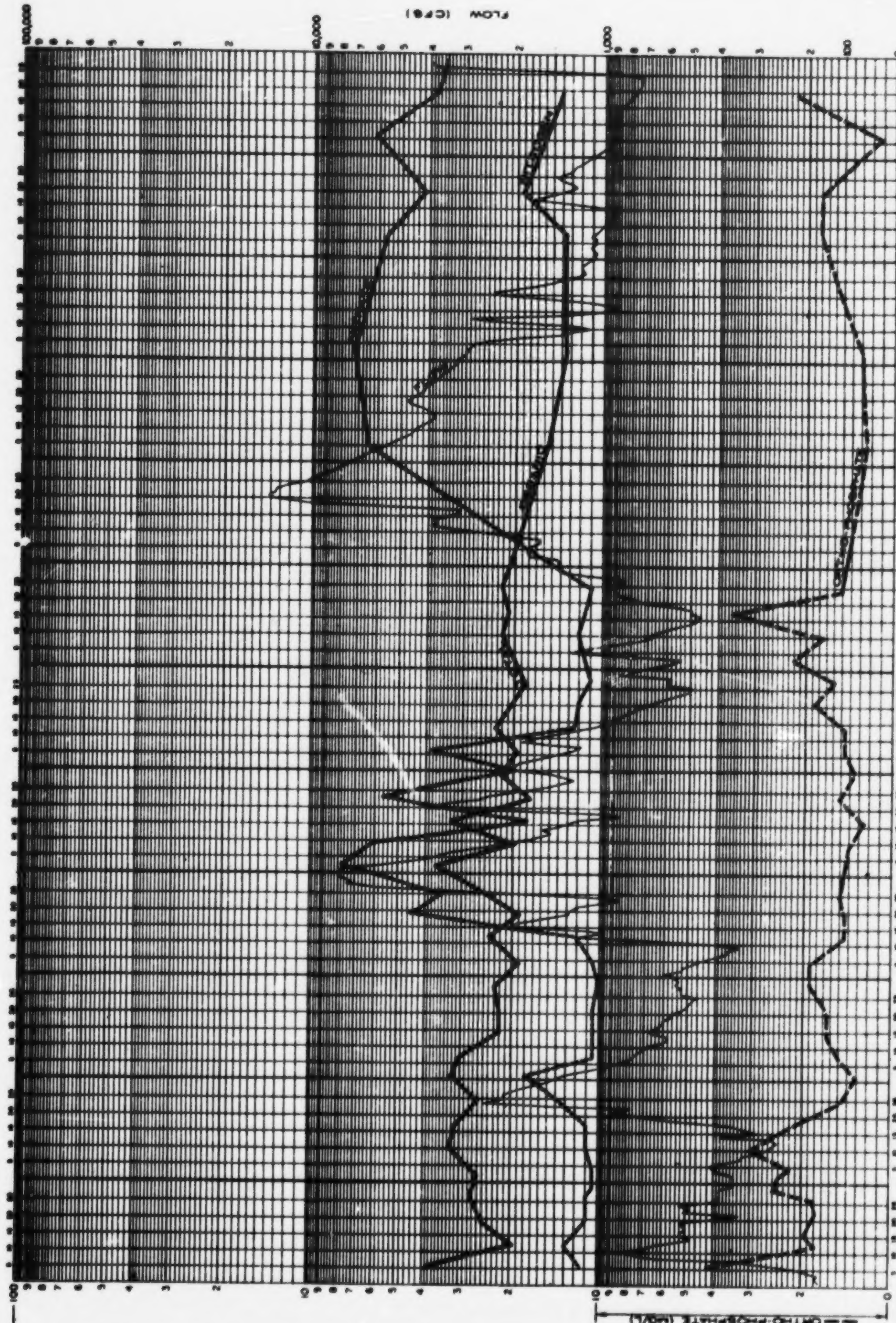
FIGURE 77



STATION R-6 DES MOINES RIVER AT HIGHWAY 46

B.O.D. & D.O. DATA

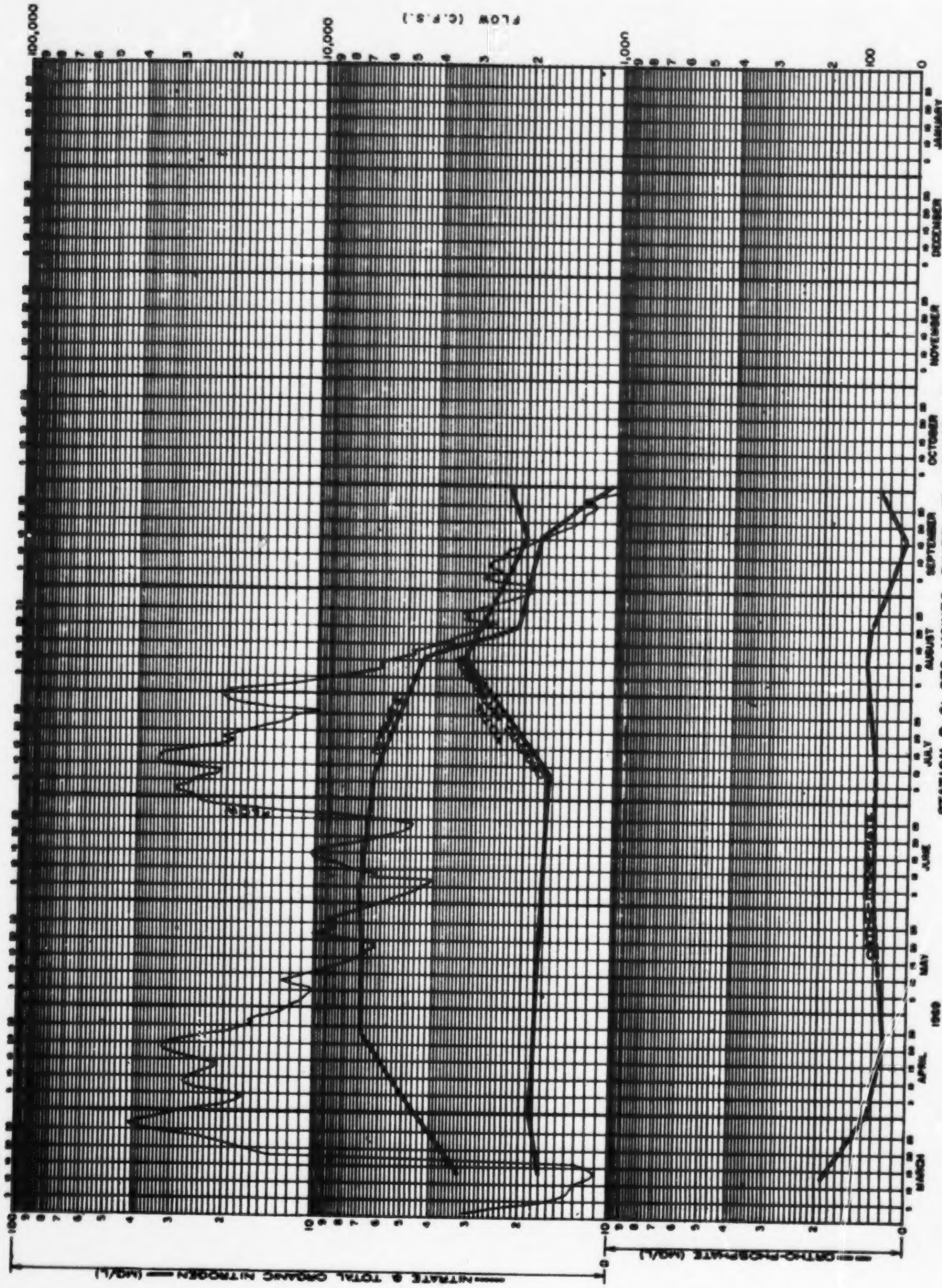
FIGURE 77 CON'T



STATION R-6: DES MOINES RIVER AT HIGHWAY 46

PHOSPHATE & NITROGEN DATA

FIGURE 78



STATION R-6: DES MOINES RIVER AT HIGHWAY 46

PHOSPHSATE & NITROGEN DATA

FIGURE 78 CON'T

APPENDIX D

DESIGN AND ASSEMBLY OF BUBBLER-TYPE LIQUID LEVEL RECORDER

Bubbler-type liquid level recorders were designed and fabricated by the Contractor as a part of this project. This was done to provide a recording device which meetsthe particular requirements of the monitoring program.

The requirements were as follows:

1. The unit must be capable of recording a wide range of liquid levels in sanitary, storm, and combined sewers.

2. The unit would be placed in a weather-proof enclosure and located at street curb, outside in remote locations, and in sewer manholes.

3. The unit must be portable and self-sufficient. External power would not be available.

4. The unit must be vandal-proof.

5. The unit should have no moving parts in the monitored flow and should have no significant obstruction in the flow pattern.

6. The unit should be capable of being installed at some distance from the monitored point.

7. The unit should not be subject to damage from freezing or ice cover.

The concept selected was a bubbler-type installation used frequently for liquid level recording in wastewater treatment facilities and pumping stations. The gas supply would be from high-pressure cylinders using either dry air or carbon dioxide. A pressure recorder, regulating valves, and gas supply would be enclosed in a water-proof and vandal-proof case, small enough to be portable. Plastic or other flexible tubing would be used to convey the gas from the unit to the point of monitoring.

Figure 79 contains a sketch of the unit with identification of the major components. The unit and typical installations are pictured in Figure 7 and numerous other figures throughout the text of this report.

The basic components of the bubbler unit are:

1. Foxboro Model 12 R 12-inch circular case pressure recorder. The recorder has a two-speed spring wound chart drive for 24-hour or 7-day operation and simultaneously operating high and low level recording elements (0 to 30" and 0 to 300" of water) with over-range protection for the low range element.

2. Foxboro Purge Rotometer, Model D-105-NX, manually adjustable to supply air or CO_2 at 0.2 to 2.0 CFH with a metering tube for visual indication of flow.

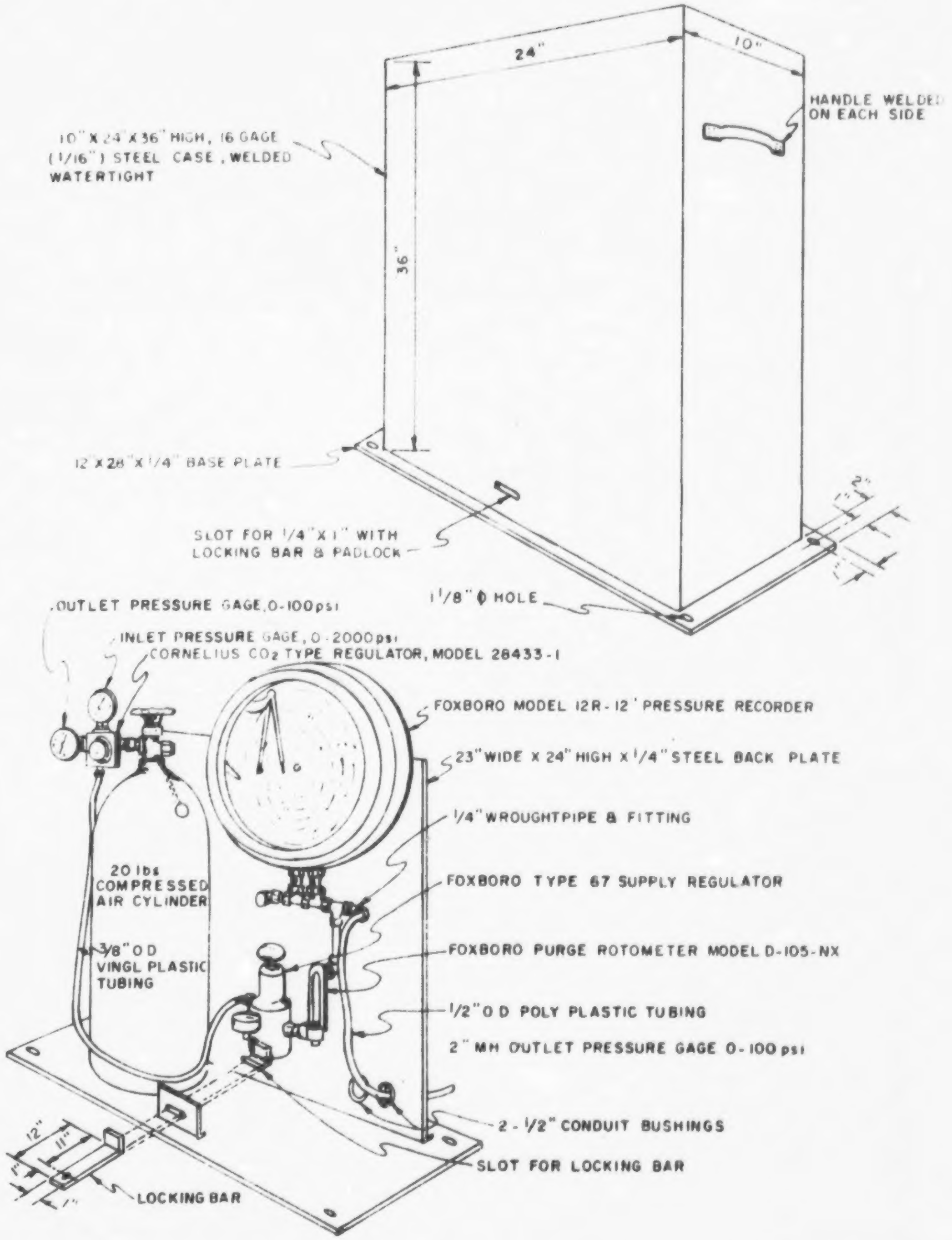
3. Foxboro Type 67 supply regulator, with 2" MH outlet pressure gage. Maximum supply pressure is 250 psi and range of control pressures (outlet pressure) is 2 to 50 psi.

4. Cornelius CO_2 type step-down Regulator, Model 28433-1, with 0 to 2000 psi inlet pressure gage and 0 to 100 psi outlet pressure gage, for initial pressure reduction from air cylinder.

5. Compressed air cylinders, 20 lb. size, filled with dry air to 1800 to 2000 psi.

6. Vandal-proof steel case, 10" deep by 24" wide by 36" high.

In the interest of minimizing the time required for assembly and delivery of the units, the enclosure was fabricated and the unit assembled by local shops. Metal piping was 1/4" wrought iron pipe. Mounting of step-down regulator directly to the compressed air cylinder permitted the use of high pressure plastic tubing connection to the supply regulator. The case was provided with holes in the base for securing the unit and an opening in the back for the bubbler line. Where vandal-proofing was required, the plastic bubbler line was encased in aluminum electrical conduit.


24"
10"
HANDLE WELDED ON EACH SIDE
10" X 24" X 36" HIGH, 16 GAGE (1/16") STEEL CASE, WELDED WATERTIGHT
36"
12 X 28" X 1/4" BASE PLATE
SLOT FOR 1/4" X 1" WITH LOCKING BAR & PADLOCK
2"
1"
1"
1 1/8" ϕ HOLE
OUTLET PRESSURE GAGE, 0-100psi
INLET PRESSURE GAGE, 0-2000psi
CORNELIUS CO2 TYPE REGULATOR, MODEL 28433-1
FOXBORO MODEL 12R-12' PRESSURE RECORDER
23" WIDE X 24" HIGH X 1/4" STEEL BACK PLATE
1/4" WROUGHTPIPE & FITTING
20 lbs COMPRESSED AIR CYLINDER
FOXBORO TYPE 67 SUPPLY REGULATOR
3/8" O D VINGL PLASTIC TUBING
FOXBORO PURGE ROTOMETER MODEL D-105-NX
1/2" O D POLY PLASTIC TUBING
2" MH OUTLET PRESSURE GAGE 0-100 psi
2 - 1/2" CONDUIT BUSHINGS
12"
11"
1"
1"
SLOT FOR LOCKING BAR
LOCKING BAR

BUBBLER - TYPE LIQUID LEVEL RECORDER

FIGURE 79